

# 1 Using Ground-Penetrating Radar Methods to Investigate Reinforced Concrete Structures

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## 13 Abstract

14 This paper provides an overview of the existing literature on the subject of Ground-Penetrating Radar (GPR) methods for  
15 the investigation of reinforced concrete structures. An overview of the use of concrete and reinforced concrete in civil  
16 engineering infrastructures is given. A review of the main destructive and non-destructive testing (NDT) methods in the  
17 field is presented, and an increase in the use of GPR to reinforced concrete structures is highlighted. It was also observed  
18 that research in some application areas has been predominantly or exclusively carried out at a laboratory scale, and that  
19 similarly, other more application-oriented research has been developed only on real-life structures. The effectiveness of  
20 GPR in these areas is demonstrated. Furthermore, a case study is presented on a new methodological and data processing  
21 approach for the assessment of reinforced concrete structures using a high-frequency dual-polarised antenna system.  
22 Results have proven the advantages of using the proposed methodology and GPR system in order to improve the  
23 detectability of rebars, including secondary bottom lines of reinforcement. The horizontal polarisation was proven to be  
24 more stable compared to the vertical. Finally, it has been demonstrated that a more accurate location of the rebars in a  
25 high-density grid mesh arrangement can be obtained by means of data migration processing with a scan spacing of 5 cm  
26 and wave velocity information through the use of the hyperbola fitting method from at least the 30% of targets.

27

28 **Keywords:** ground-penetrating radar (GPR); reinforced concrete structures; non-destructive assessment; standard test  
29 methods in concrete; data sampling methodology for migration; rebar location

## 30 1. INTRODUCTION

31 Reinforced concrete is a dominant structural material in construction engineering in many countries. The extensive use  
32 of reinforced concrete structures depends on a number of factors, such as i) a wide availability of reinforcing bars and  
33 concrete constituents (gravel or crushed rock, sand, water, and cement); ii) a relative ease in concrete construction; and  
34 iii) the economy of reinforced concrete compared with other construction materials.

35 Reinforced concrete is used in many types of building, in underground structures, water tanks, wind turbine foundations  
36 and towers, offshore oil exploration and production structures, dams, bridges, and also ships (Wight and MacGregor,  
37 2012).

38 A kind of concrete was first used for the construction of structures in Crete during the Minoan civilisation, about 2000  
39 B.C. This concrete-like material was not water resistant and, therefore, could not be used for exposed or underwater  
40 structures. Thousands of years later, the Pantheon's concrete dome was completed by the Romans in A.D. 126. The  
41 building still stands as the largest unsupported dome in the world with a diameter of 49 m, and is built with concrete made  
42 with broken bricks as aggregates.

43 In 1824, Portland cement was developed by Joseph Aspdin (Sharp, 2006) who mixed ground limestone and clay from  
44 different quarries and used a kiln to produce the cement. However, the material currently known as Portland cement was  
45 created by I. C. Johnson in 1845. He found that the best cement resulted from the grinding of a hard clinker, produced  
46 when the mixture was overheated. In 1854, William Boutland Wilkinson of Newcastle patented a reinforced concrete  
47 floor (Moussard et al., 2017). The introduction of reinforcing bars allowed to cover the structural gaps of non-reinforced  
48 concrete, which is strong in terms of compression but weak in terms of tension. In fact, it is known that tensile stresses  
49 caused by loads, restrained shrinkage, or temperature changes create cracks when the tensile stress of a material is  
50 exceeded. In addition, an unreinforced beam can collapse very rapidly at the formation of first cracks. Conversely,  
51 reinforcing bars embedded in the concrete allow tension forces to form for moment equilibrium after the concrete is  
52 cracked. Prestressed concrete was introduced by Eugène Freyssinet more recently, in 1928, by putting reinforcement (i.e.  
53 pre-stressing tendons) into tension and concrete into compression (Billington, 2004). This allowed to delay the formation  
54 of cracks in the beam.

55 The construction of reinforced concrete members must follow a dedicated manufacturing process (Neville and Brooks,  
56 1987). Reinforced concrete structures consist of a series of "members" that interact to support the loads placed on the  
57 structure. An overview of the various types of configuration and load transmission modes for these structures has been  
58 given by Wight and MacGregor (2012). The advantages of using reinforced concrete in place of other construction  
59 materials such as steel, masonry, or timber have been a subject of debate. The main benefits and disadvantages of using  
60 concrete structures are summarised in Tables 1 and 2 respectively.

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**Table 1** Advantages of concrete structures

Consideration	Assessment
Availability	The foremost consideration is the wide availability of reinforcing bars and of the concrete constituents (gravel or crushed rock, sand, water, and cement) that can be easily found and transported to the construction site and, hence, it is preferred as a construction material in remote areas.
Economy	From an overall structural cost point of view, reinforced concrete constructions are cheaper compared to other construction materials in terms of cost of materials and the relative ease in concrete construction.
Suitability	From a shape point of view, concrete has the advantage that it is placed in plastic conditions and it allows to give the desired shape according to the structural function.
Fire resistance	Concrete buildings are fire resistant, with 1- to 3-hour fire rating without special fireproofing.
Rigidity	Concrete buildings have greater stiffness and mass that prevent the formation of oscillations and vibrations.
Low maintenance	Concrete buildings require less maintenance than buildings made of structural steel or timber members.

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**Table 2** Drawbacks of concrete structures

Consideration	Assessment
Low tensile strength	Concrete is strong in compression and weak in tension. The tensile strength is about 1/10 of the compressive strength, therefore reinforcements are required when concrete is subject to tensile stresses.
Low strength per unit of weight or volume	Concrete has a compressive strength and a unit density of about 10% and 30% the steel, respectively. Therefore, a concrete structure requires a larger volume and a greater weight than a steel structure.
Forms	The construction process of concrete structures involves labour and time due to the use of more complex formworks and finishes compared to other types of structures.
Volume changes	At the curing stage, concrete is subject to a drying shrinkage that may cause cracking or unwanted deflections.
Energy used in manufacture	Cement used for concrete production is a high-energy material that results in high costs.
Environmental impact	<ul style="list-style-type: none"> <li>• The high levels of CO<sub>2</sub> emissions at the production stage cover the 5% of the worldwide generation of CO<sub>2</sub>.</li> <li>• Loss of agricultural lands as well as visual impact caused by major concrete structures on some areas of outstanding natural beauty are elements of major concern.</li> <li>• Care is necessary for the extraction of natural raw material components to avoid damage to the landscape.</li> </ul>

65

66 A number of key processes may damage the integrity of a concrete structure, especially during the material's service life.

67 Corrosion associated with reinforcing bars is the most significant factor contributing to structural damage (Beena et al.,

68 2017). It is usually caused by excessive moisture content and chloride ions in the parts of the concrete adjacent to the

69 reinforcing steel. The early stage of corrosion is generally known as the initial passivity stage. Reinforcing bars are rapidly  
70 attacked by acidic, wet corrosion, yield expansive, hydrated corrosion products. These can in turn accelerate the formation  
71 of cracks in the surrounding concrete. The oxidation products occupy a larger volume than the original intact steel and so  
72 the internal expansive stresses lead to cracking and debonding. This occurrence is commonly known as the delamination  
73 of concrete cover material from the reinforcing layer (Neville and Brooks, 1987; Meng et al., 2020).

74 Within this framework, a comprehensive assessment and monitoring of concrete structures is required in order to properly  
75 identify the type and stage of decay. Traditional methods for investigating reinforced structures rely mostly on the use of  
76 destructive techniques (Tosti and Slob, 2015). In this regard, the most frequently used methods are coring, drilling or  
77 otherwise removing part of the structure to allow inner visual inspections of rebar conditions. Although destructive  
78 techniques provide accurate information, they are expensive, time-consuming and significant only at the point of  
79 acquisition. In addition, local damage at the sampling point may spread and can accelerate the process of decay.

80 In view of the above, non-destructive testing (NDT) methods are gaining momentum in the assessment of civil  
81 engineering structures and infrastructures as fundamental tools for providing a more comprehensive survey programme  
82 (Loizos and Plati, 2007; Plati et al., 2014; Fontul et al., 2018). NDT methods have taken hold in the field of concrete  
83 structures in view of the increasing accuracy and effectiveness of their measurements.

84 There exists a wide spectrum of NDT techniques based on different theoretical principles and the production of various  
85 sets of information regarding the physical properties of a structure. In general, it is worth considering five major factors  
86 for the planning of a non-destructive survey (McCann and Forde, 2001). These are i) the required depth of penetration  
87 into the structure; ii) the vertical and lateral resolution required for the anticipated targets; iii) the contrast in the physical  
88 properties between the target and its surroundings; iv) the signal-to-noise ratio for the physical property measured on the  
89 structure under investigation; v) the historic information concerning the methods used in the construction of the structure.

90 The major NDT methods used for the investigation of reinforced concrete structures are the sonic/ultrasonic methods, the  
91 infrared thermography, radiography and the electromagnetic (EM) methods (e.g. radar-based methods).

92 Non-destructive sonic/ultrasonic testing methods have been used for the past thirty years for the assessment of civil  
93 engineering structures and materials (McCann and Forde, 2001). These methods are based on the transmission and  
94 reflection of mechanical stress waves at sonic and ultrasonic frequencies through a medium. In more detail, a sonic  
95 transmission method involves the passing of a compression wave at frequencies between 500 Hz and 10 kHz, generated  
96 by the impact of a force hammer with the structure. Effects of the wave are received by an accelerometer located on the  
97 opposite side of the force hammer. An improvement of the sonic method is sonic tomography, where tests are not  
98 restricted to being carried out along a path perpendicular to the surface of the medium. In the sonic reflection method,  
99 both the initiation and the reception stage of a wave are performed on the same face of the medium's surface. Finally,

100 ultrasonic methods rely on the use of ultrasonic waves at frequencies above 20 kHz. This method was the first non-  
101 destructive technique developed for the testing of concrete. Nevertheless, the ultrasonic method is less practical in  
102 concrete structures due to the higher attenuation characteristics of concrete and hence lower frequency signals are needed  
103 to obtain a reasonable penetration. Another major issue with using ultrasonic methods is the scattering of both incident  
104 and reflected waves, due to the different types of materials used in the mix.

105 Infrared thermography is a technique that allows heat at any temperature to be converted into a thermal image using a  
106 specific scanning camera (Stanley and Balendran, 1995). With this technology, it is possible to assess buildings or  
107 structures with defects (e.g. debonding render and mosaic or delaminating concrete) with regards to different infrared  
108 radiations. A concrete surface that is free of defects appears as relatively uniform in a thermal camera. On the other hand,  
109 cracks or other surface defects are displayed in red as they heat up faster under solar irradiation.

110 Radiography is another NDT method that uses x-rays, gamma rays and neutron rays, which are types of very short  
111 wavelength EM radiations. It is based on the detection and recording of the wave radiation penetrating through a medium.  
112 The amount of absorption depends on the density and thickness of the material as well as on the characteristics of the  
113 radiation. The radiation passing through the medium is recorded on a film or a sensitised paper and viewed on a screen.  
114 The sensitivity of the radiography depends on several factors, as the thickness and the direction of the radiation with  
115 respect to the defect: the best results can be obtained when a defect is significantly thick in a parallel direction to the  
116 radiation beam. The main application of radiographic techniques is the welds and castings inspection (Mishin, 1997).

117 EM methods are based on electromagnetic theory, described by the Maxwell's equations, and the material properties,  
118 quantified by constitutive relationships. These methods use the radar technology that is being spread quickly among NDT  
119 methods in the field of structural engineering, including applications on reinforced concrete structures.

120 In more detail, ground-penetrating radar (GPR) is a geophysical inspection method used to scan the subsurface in many  
121 application areas (Lai et al., 2009), including concrete structures such as buildings, dams (Rhim, 2001) and bridges (Kohl,  
122 2006). GPR is a relatively new geophysical technique that has seen major advances in the last decade (Gizzi and Leucci,  
123 2018), especially for civil engineering applications (Annan, 2004; Daniels, 2004). The working principle of the GPR  
124 technique is based on sending a short-duration EM wave and recording arrival time, amplitude and phase of the back-  
125 reflected signal. These outputs are dependent on the GPR characteristics and the properties of the host material.

126 According to Burgey et al. (2003), GPR has several key practical advantages as opposed to other geophysical NDT  
127 methods. It is in fact a piece of light and portable equipment where no coupling medium or special safety precautions are  
128 required, in terms of emission of microwaves. In addition, GPR allows some of the most critical features in reinforced  
129 concrete structures to be detected.

130 The present work reports an overview of the use of GPR methods for the investigation of reinforced concrete structures.  
131 Section 2 discusses the main international standard test methods and guidelines. The theoretical background and working  
132 principles of the GPR technology with an overview of the main applications in reinforced concrete structures are given  
133 in Section 3. Section 4 reports an historical review of research methods on the use of GPR in this subject area. Section 5  
134 presents a case study on a new methodological and data processing approach for the assessment of reinforced concrete  
135 structures. Conclusions are presented in the last section of the paper.

136

## 137 **2. STANDARD TEST METHODS AND GUIDELINES**

138 This Section reports a selection of principal destructive, partially-destructive and non-destructive standard test methods  
139 for concrete and reinforced concrete assessment. Discussed standards have been released by the American Society for  
140 Testing and Materials (ASTM), the American Concrete Institute (ACI) and the British Standards Institution (BSI).

141 Destructive testing methods require samples of the investigated structural element to be collected, usually in the form of  
142 cores. These are assessed in the laboratory for strength analyses and other physical tests as well as visual, petrographic  
143 and chemical investigations. Partially-destructive testing methods include tests requiring a minor level of damage to the  
144 material or structural member investigated.

145 NDT methods are generally defined as those methods which do not impair the intended performance of an element or a  
146 structural member under test conditions. In regard to concrete, NDT techniques are intended to include also methods  
147 causing localised surface damage. The main advantage of these methods is that they can be performed on the concrete  
148 directly on site without the need to collect samples.

149

### 150 **2.1. Destructive and Partially-Destructive Methods**

151 In this Section, use of cores is discussed as the major destructive testing method for concrete. In regard to the use of  
152 partially-destructive methods, surface hardness methods are presented for the assessment of the quality of concrete. In  
153 addition to these, the penetration resistance method, the pull-out and pull-off and the break-off testing methods are  
154 discussed in regard to the evaluation of the strength of concrete.

155

#### 156 ***2.1.1. Destructive Testing Methods***

157 Coring is the most popular and established destructive way of testing concrete, as it allows visual inspections of the inner  
158 regions of a member to be coupled with strength estimations. Standards recommending procedures for cutting, testing  
159 and interpretation of results are available worldwide (e.g., BS EN 12504-1, 2009; ASTM C42, 2018); ACI 318, 2014).

160 The equipment usually employed for coring is a rotary cutting tool with diamond bits (Fig 1). To avoid drilling of distorted  
161 or broken cores, the equipment must be firmly supported and braced against the concrete to prevent relative movements.  
162 It is also necessary to provide a water supply in order to lubricate the cutter.  
163 It is important to plan core location e size before testing. Core location is primarily linked to the basic purpose of the  
164 testing. Regarding the core size, standards require a diameter of minimum three times the nominal maximum aggregate  
165 size. In general, the accuracy decreases as the ratio of the aggregate size to the core diameter increases. 100 mm diameter  
166 cores should not be used if the maximum aggregate size exceeds 25 mm, whereas 75 mm cores should be used preferably  
167 for aggregate size of less than 20 mm. r.  
168 Two basic classes of factors affecting measured core compressive strength can be mentioned. These are related to the  
169 concrete characteristics and the testing variables. Regarding the concrete characteristics, the moisture condition of the  
170 core affects the measured strength. Therefore, it is very important that the relative moisture conditions of the core and the  
171 in-situ concrete are considered for the estimation of the actual concrete strengths. Voids in the core are also an important  
172 factor capable to reduce the measured strength.  
173 In regard to the testing variables, many different factors have been observed to affect the measured strength, i.e., i)  
174 length/diameter ration of the core; ii) diameter of the core; iii) direction of drilling; iv) method of capping; v)  
175 reinforcement.



176  
177 **Fig. 1** A core cutting drill

178  
179 **2.1.2. Partially-Destructive Methods**

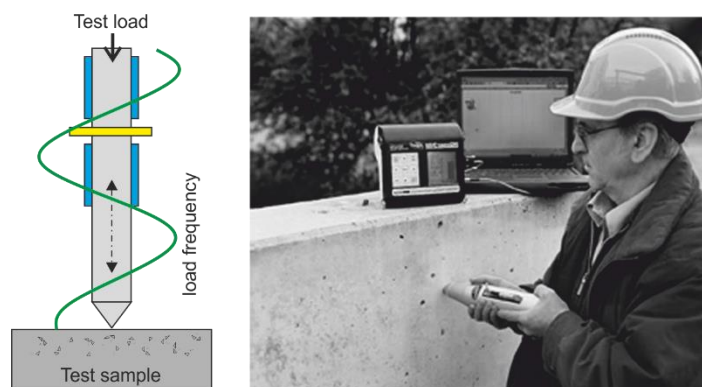
180 *Surface Hardness Methods*

181 Hardness is a key factor to test within the context of assessing the quality of concrete. Surface hardness methods can be  
182 used in several application areas in order to assess the uniformity of concrete quality and compare the outcomes with

specific requirements. Furthermore, these methods can provide a reliable estimate of the concrete strength and resistance. First applications of measuring the surface hardness of a mass of concrete were recorded in the 1930s. They involved measurements of the size of indentation caused by a steel ball either fixed to a pendulum or a spring hammer, or fired from a standardised testing pistol. The height of rebound of the mass from the surface was then measured at a later stage. The purpose of these methods is not to establish a theoretical relationship between the values measured from any of these methods and the strength of concrete, but rather to define empirical relationships between test results and the quality of the surface layer.

The equipment is a rebound test hammer developed by the Swiss engineer Ernst Schmidt in the late 1940s (Fig. 2). The spring-controlled hammer mass slides on a plunger within a tubular housing. The plunger is pressed strongly and steadily against the concrete at right angles to its surface, until the spring-loaded mass is triggered from its locked position. After the impact, the scale index is read while the hammer is still in the test position. To avoid local variations of the values in the concrete due to irregularity of the surface, it is necessary to collect several readings for each test, typically nine spaced 25 mm from each other in an area not exceeding 300 mm<sup>2</sup>. Therefore, the surface must be smooth, clean and dry.

According to Bungey et al. (2003) several factors can affect the results. Some of these factors are related to the mix characteristics, i.e., i) cement type, ii) cement content, and iii) coarse aggregate type; whereas other factors relates to the member characteristics, i.e., i) mass, ii) compaction, iii) surface type, iv) age, v) surface carbonation, vi) moisture condition, vii) stress and temperature. Use of the rebound method is regulated by BS EN 12504-2 (BS EN 12504-2 2013) and ASTM C805 (ASTM C805 2018).



**Fig. 2** The surface hardness test method (Bungey et al. 2006)

### *Penetration Resistance Testing*

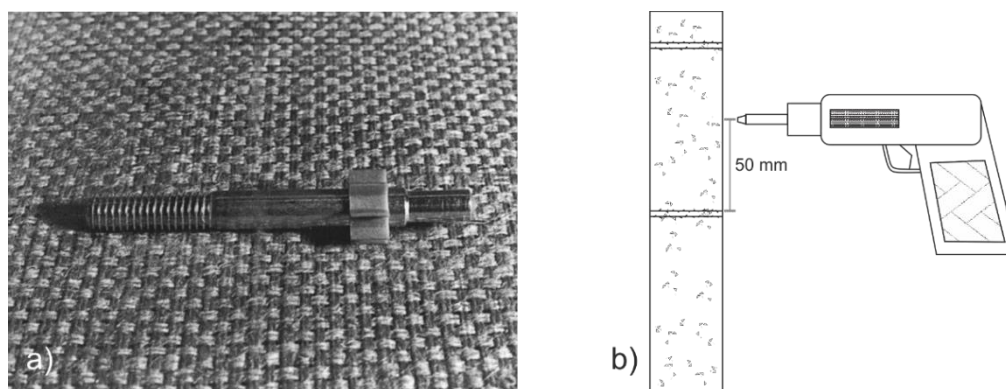
Penetration resistance testing (Fig. 3) is based on the principle that the depth of penetration is influenced by the strength of the concrete. A first application, known as the Windsor probe test, was developed in the US during the mid-1960s.



208 Although it is difficult to provide a theoretical relationship between the depth of penetration of the bolt and the concrete  
 209 strength, robust empirical relationships can be found that are virtually not affected by the operator technique.

210 The test equipment consists in probes, generally of 6.35 mm in diameter and 79.5 mm in length, that can work alone or  
 211 in groups of three. A system of triangular measuring plates is used to provide one averaged reading of the exposed length  
 212 for the group of probes. The test procedure recommends to perform at least three tests and to provide an averaged outcome.  
 213 In case the range of a group of three tests exceeds 5 mm, a further test should be performed and the extreme value  
 214 discarded.

215 The principal physical limitation of this method is on the need of providing an adequate edge distance and a member  
 216 thickness of at least twice the size of the anticipated penetration. Recommendations on the use of penetration resistance  
 217 testing are given in BS 1881-207 (BS 1881-207 1992) and ASTM C803 (ASTM C803 2018).



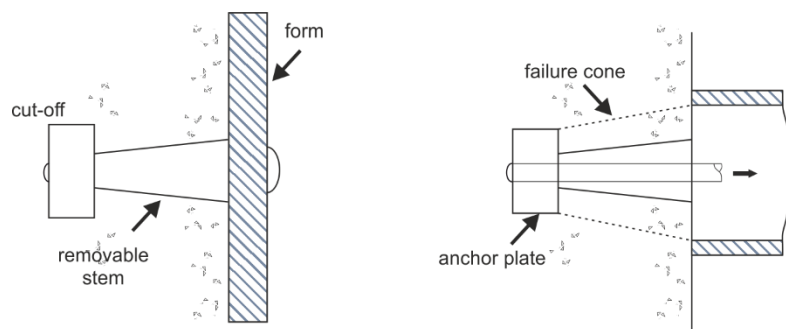
219  
 220 **Fig. 3** Penetration resistance testing. **a** Penetration resistance test probe (Bungey et al. 2006), **b** layout of test procedure

## 222 *Pull-Out and Pull-Off Testing Methods*

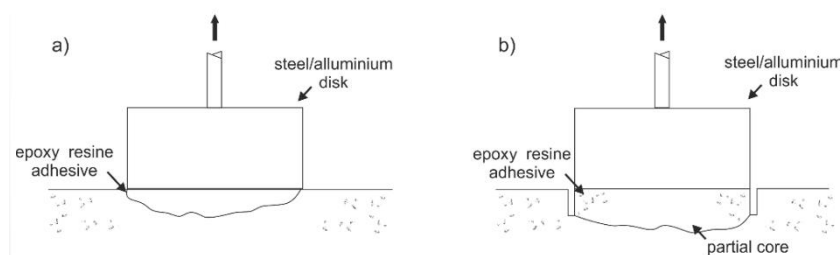
223 Pull-out testing methods are based on measuring the force required to pull a bolt or other similar metal bars from a  
 224 concrete surface. Tests fall into two classes, i.e., the cast-in and the drilled-hole methods. Cast-in methods involve an  
 225 insert casted into the concrete, whereas drilled-hole methods provide an insert fixed into a hole which is drilled into the  
 226 hardened concrete. In both cases, the value of the test is dependent on the ability to relate the pull-out force to the concrete  
 227 strength. It has also been observed that this relationship is relatively independent from concrete mix features and curing  
 228 history (Carino, 2004). An example of cast-in methods, i.e., the Lok-test, is illustrated in Fig. 4.

229 Pull-off methods (Fig. 5) have been developed to measure the in-situ tensile strength of concrete by applying a direct  
 230 tensile force. Procedures are covered by BS 1881-207 (BS 1881-207 1992), ASTM C1583 (ASTM C1583 2013) (in-situ  
 231 applications) and BS EN 1542 (BS EN 1542 1999) (laboratory applications on specimens) standards to assess the bond  
 232 properties of repairing materials. These methods were developed initially in the early 1970s. Working principle provides

233 a disk to be glued to the concrete surface with an epoxy resin. The disk is jacked off to measure the force necessary to  
 234 pull a piece of concrete away from the surface. A nominal tensile strength for the concrete is calculated based on the disk  
 235 diameter. The compressive strength can be also calculated using a calibration chart appropriate to the concrete.  
 236



237  
 238 **Fig. 4** Pull-out methods: the Lok-test insert  
 239



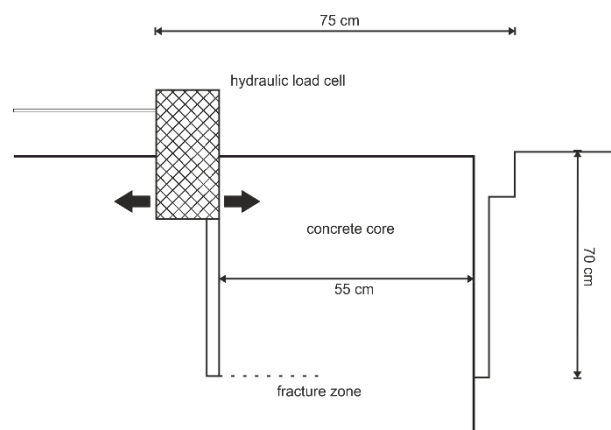
240  
 241 **Fig. 5** The pull-off method. **a** Surface, **b** partially cored  
 242

### 243 *Break-Off Testing Methods*

244 The break-off technique has been developed following three different procedures. The procedure developed in Norway  
 245 is known as “Norwegian method” (Ottosen, 1981) and it allows to determine directly the flexural strength in a plane  
 246 parallel to and spaced from the concrete surface. The configuration of the test provides a tubular disposable form inserted  
 247 into the fresh concrete. A transverse force is applied at the top surface in order to break the core left after the removal of  
 248 the insert (Fig. 6). Specially developed portable equipment are used in order to exert hydraulically the breaking force. A  
 249 linear correlation has been found between the break-off strength and the rupture modulus measured on prism specimens.  
 250 The entire procedure and recommendations for this testing method are detailed in ASTM C1150 (ASTM C1150 2002).  
 251 Another procedure known as the Stoll tork test (Stoll, 1985), was proposed later on in order to improve upon the variability  
 252 of the existing similar techniques and to allow for testing at deeper depths than the pull-out, the pull-off and the penetration  
 253 resistance methods. Accuracy of this testing method is comparable with that of the major partially-destructive methods,  
 254 with results based on the average of at least three tests. The main value of the method is in the pre-planned monitoring of

the internal in-situ strength development, although further investigations are still required to foster a commercial use of the method.

A further test procedure for precast concrete quality control purposes has been used in the former Soviet Union, as discussed by Leshchinsky et al. (1990). However, this technique has not become an established international method for testing.



**Fig. 6** The break-off method

## 2.2. Non-Destructive Methods

Several applications of NDT methods to concrete and reinforced concrete materials and structures can be found in the literature. Although research in this subject area is continuously in progress and use has not become established for many of these methods, promising results have been achieved to date. Table 3 reports an overview of the main NDT techniques used for concrete and reinforced concrete investigation purposes, highlighting their main advantages, limitations, areas of future research, and potential future challenges and prospects.

**Table 3** Main NDT methods for investigation of concrete and reinforced concrete materials and structures

Method	Standard/Guideline	Characteristic	Theoretical frequency range	Application	Advantage	Limitation	Future research challenge and prospect
Sonic	<ul style="list-style-type: none"> <li>ASTM C597-16 (2016)</li> </ul>	<ul style="list-style-type: none"> <li>Contact</li> <li>Partially-destructive</li> </ul>	500 Hz ÷ 10 KHz	<ul style="list-style-type: none"> <li>Evaluation of delamination (McCann and Forde, 2001)</li> <li>Voids detection (Fegen et al., 1979)</li> <li>Assessment of material uniformity (McCann and Forde, 2001; Guida et al., 2012)</li> <li>Assessment of the average compressive strength of structures/materials (McCann and Forde, 2001)</li> </ul>	<ul style="list-style-type: none"> <li>Reliability of results (McCann and Forde, 2001)</li> <li>High-resolution imaging (Hollema and Olson, 2003)</li> <li>Suitability for outdoor surveys (McCann and Forde, 2001)</li> <li>Useful information on major structural elements (Guida et al., 2012)</li> </ul>	<ul style="list-style-type: none"> <li>Invasiveness if electrical probes are needed (ACI 228.2R-13, 2013)</li> <li>Complexity of result interpretation (McCann and Forde, 2001; ACI 228.2R-13, 2013)</li> <li>High signal attenuation for high-resolution imaging (McCann and Forde, 2001)</li> <li>Time-consuming data collection (ACI 228.2R-13, 2013)</li> <li>Moderately high costs (McCann and Forde, 2001)</li> </ul>	<ul style="list-style-type: none"> <li>Development of more user-friendly devices for data acquisition</li> <li>Use of an array of transducers to limit data acquisition time</li> </ul>

Ultrasonic	<ul style="list-style-type: none"> <li>ASTM C597-16 (2016)</li> <li>ASTM D6760-16 (2016)</li> <li>BS EN 12504 (2004)</li> </ul>	<ul style="list-style-type: none"> <li>Contact</li> <li>Partially-destructive</li> </ul>	20 ÷ 200 KHz	<ul style="list-style-type: none"> <li>Reinforcement detection (Pucinotti et al., 2007)</li> <li>Location of a variety of defects within concrete elements (e.g., delaminations, voids, honey-combing) (ACI 228.2R-13, 2013)</li> <li>Thickness evaluation of structural elements (ACI 228.2R-13, 2013)</li> <li>Assessment of the relative condition or uniformity of concrete (ACI 228.2R-13, 2013)</li> </ul>	<ul style="list-style-type: none"> <li>Portable equipment is available (ACI 228.2R-13, 2013)</li> <li>Relatively easy to use (ACI 228.2R-13, 2013)</li> <li>High speed of tests (Bungey et al., 2006)</li> <li>Reliability of results (Bungey et al., 2006)</li> </ul>	<ul style="list-style-type: none"> <li>No information on major elements (Bungey et al., 2006)</li> <li>Applicable to limited member thickness (ACI 228.2R-13, 2013)</li> <li>Experienced operators required (ACI 228.2R-13, 2013)</li> </ul>	<ul style="list-style-type: none"> <li>Possibility to reduce signal attenuation effects</li> </ul>
GPR	<ul style="list-style-type: none"> <li>ASTM D4748-15 (2015)</li> <li>ASTM D6432-11 (2011)</li> <li>ASTM D6087-08e1 (2015)</li> </ul>	<ul style="list-style-type: none"> <li>Contact/No n-contact</li> <li>Fully non-destructive</li> </ul>	500 MHz ÷ 2.5 GHz	<ul style="list-style-type: none"> <li>Defect and decay detection (Wang et al., 2011; Martino et al., 2014; Miramini et al., 2018)</li> <li>Location of rebars (Alvarez et al., 2017; Kien et al., 2018; Lachowicz and Rucka, 2018)</li> <li>Estimation of rebar size (Utsi and Utsi, 2004; Chang et al., 2009; Zhou et al., 2018; Giannakis et al., 2019)</li> <li>Measurement of dielectric properties (Louzli et al., 2002; Laurens et al., 2005; Sbartaï et al., 2009b)</li> <li>Industrial quality control (Wiwatrojanagul et al., 2017; Bungey et al., 2003)</li> </ul>	<ul style="list-style-type: none"> <li>Totally non-destructive (Zhou et al., 2018)</li> <li>Portable equipment (Daniels, 2004)</li> <li>Use of different frequencies for different types of targets (Bungey et al., 2006)</li> <li>Real-time continuous displaying of collected results (Miramini et al., 2018)</li> <li>Rapid investigations of large areas (Bungey et al., 2003)</li> <li>Very sensitive to presence of embedded metal objects (ACI 228.2R-13, 2013)</li> <li>Sensitive to the presence of moisture and chlorides (ACI 228.2R-13, 2013)</li> </ul>	<ul style="list-style-type: none"> <li>Skills required to interpret data (Bungey et al., 2003)</li> <li>Congested reinforcement can prevent penetration beyond the reinforcement (ACI 228.2R-13, 2013)</li> <li>Difficulty in detecting early-stage decay (Bungey et al., 2003)</li> <li>Cracks and delaminations not easy to detect unless moisture is present in the cracks or in the region of the delamination (ACI 228.2R-13, 2013)</li> <li>Limited depth of penetration of pulses from high-resolution antennas (300 to 500 mm) (ACI 228.2R-13, 2013)</li> </ul>	<ul style="list-style-type: none"> <li>Improving existing knowledge on the behaviour of the EM pulses through reinforced concrete structures</li> <li>Use of higher central frequencies for detection of early decay</li> <li>Development of real-time algorithms for concrete investigation</li> </ul>
Microwaves	/	<ul style="list-style-type: none"> <li>Non-contact</li> <li>Fully non-destructive</li> </ul>	250 MHz ÷ 300 GHz	<ul style="list-style-type: none"> <li>Concrete decay (Patriarca et al., 2011; Hashemi, 2016)</li> <li>Moisture distribution (Rhim and Buyukozturk, 1998; Bouichou et al., 2018)</li> </ul>	<ul style="list-style-type: none"> <li>Small size of the antennas (Hashemi, 2016)</li> <li>High-resolution measurements (Bouichou et al., 2018)</li> </ul>	<ul style="list-style-type: none"> <li>Available hardwires not suitable for outdoor surveys (Hashemi, 2016)</li> <li>Difficulty to identify the nature of decay (Hashemi, 2016)</li> </ul>	<ul style="list-style-type: none"> <li>Better understanding of the physical properties of decays</li> <li>Additional research required with computer analysis and sensor technology</li> <li>Need for more quantitative results</li> </ul>
Infrared Thermography	<ul style="list-style-type: none"> <li>ASTM D4788-03, (2013)</li> </ul>	<ul style="list-style-type: none"> <li>Non-contact</li> <li>Fully non-destructive</li> </ul>	3 THz ÷ 428 THz	<ul style="list-style-type: none"> <li>Voids and delamination detection (McCann and Forde, 2001)</li> <li>Defect evaluation (Holt and Eales, 1987)</li> <li>Assessment of concrete moisture conditions (Grinzato et al., 2011)</li> <li>Construction details (Titman, 2001)</li> <li>Detection of reinforcing bars (Maierhofer et al., 2006)</li> </ul>	<ul style="list-style-type: none"> <li>Reliability of results (Stanley and Balendran, 1995)</li> <li>Suitability for rapid assessment of large or high-rise buildings (Stanley and Balendran, 1995)</li> <li>Remote use without direct coupling with structure/material (ACI 228.2R-13, 2013)</li> <li>Good overview of decay areas in the survey region (ACI 228.2R-13, 2013)</li> <li>Real-time displaying of collected results (Rocha et al., 2019)</li> </ul>	<ul style="list-style-type: none"> <li>Variations in the test response with varying environmental conditions (McCann and Forde, 2001; ACI 228.2R-13, 2013)</li> <li>Limitations in detecting deep defects (Stanley and Balendran, 1995; ACI 228.2R-13, 2013)</li> <li>Difficulty in decay detection for low-quality concrete (Rocha et al., 2019)</li> <li>Expensive equipment (ACI 228.2R-13, 2013)</li> <li>Experienced operator required (ACI 228.2R-13, 2013)</li> </ul>	<ul style="list-style-type: none"> <li>Investigation of deeper layers</li> <li>Use of special filters to reduce the effects of variable environmental conditions</li> </ul>
Radiography	<ul style="list-style-type: none"> <li>BS 1881-205:1986 (1986)</li> <li>ASTM C1040 / C1040M-16a (2016)</li> </ul>	<ul style="list-style-type: none"> <li>Contact</li> <li>Fully non-destructive</li> </ul>	$1 \times 10^4$ THz ÷ $3 \times 10^6$ THz (X-rays) $3 \times 10^6$ THz ÷ $3 \times 10^{10}$ THz (Y-rays)	<ul style="list-style-type: none"> <li>Providing view of the internal structure of the test object (Mishin, 1997; ACI 228.2R-13, 2013)</li> <li>Use of image plates to extract more information about the internal structure of the test object (ACI 228.2R-13, 2013)</li> <li>Checking of reinforced bars (Mitchell, 1991)</li> <li>Use of portable iridium-192 (Ir) sources in holes drilled in concrete to provide tomography of reinforcement in large columns and beams (ACI 228.2R-13, 2013)</li> </ul>	<ul style="list-style-type: none"> <li>Equipment can be turned off when not in use (X-rays) (ACI 228.2R-13, 2013)</li> <li>Equipment reasonably portable and cost-effective (Y-rays) (ACI 228.2R-13, 2013)</li> <li>Minimal operator skills required for data collection (Y-rays) (ACI 228.2R-13, 2013)</li> <li>Reliability of results for large datasets (Bungey et al., 2006)</li> </ul>	<ul style="list-style-type: none"> <li>Safety concerns due to the emission of hazardous radiations (ACI 228.2R-13, 2013)</li> <li>Operators must be licensed and highly skilled.</li> <li>Bulky and expensive equipment (X-rays) (Bungey et al., 2006; ACI 228.2R-13, 2013)</li> <li>Y-ray penetration limited to 20 in. (500 mm) within concrete materials (ACI 228.2R-13, 2013)</li> <li>Access to opposing faces required (ACI 228.2R-13, 2013)</li> <li>Large differences more readily detected than small differences (Mishin, 1997)</li> <li>Difficulty to identify cracks perpendicular to the radiation beam (ACI 228.2R-13, 2013)</li> </ul>	<ul style="list-style-type: none"> <li>Additional research required to deal with large data samples</li> </ul>

271

272 In regard to the existing standards for use of NDT techniques in concrete investigations, four classes of NDT methods are

273 presented in this Section which have been sorted by the main inspection task, i.e., (1) ultrasonic pulse velocity methods

274 for concrete quality control purposes; (2) durability tests for concrete decay assessment due to weathering action and

275 chemical attack; (3) performance and integrity tests and (4) load testing methods.

276

### 277 2.2.1. *Quality Control Tests: Ultrasonic Pulse Velocity Methods*

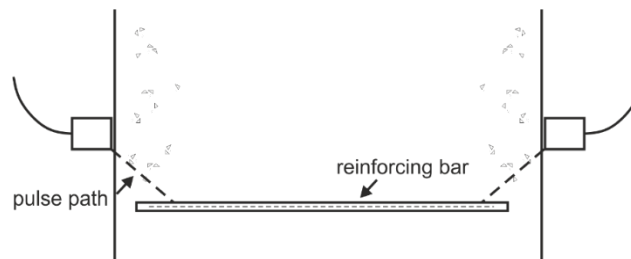
278 The first application of pulse velocity methods in concrete dates back to the mid-1940s, USA. Methods were considered  
279 reliable in view of a strong dependence of the velocity upon the elastic properties of the material. A few years later, a  
280 repetitive mechanical pulse equipment was developed in France to overcome measurement problems. Nowadays, modern  
281 ultrasonic methods, employing pulses in a frequency range of 20–150 kHz, are the most diffused.

282 In terms of working principles, a source pulse is transmitted to the concrete surface and a recorded pulse is received and  
283 amplified. This allows to measure the time elapsed with an accuracy of  $\pm 1\%$ . The wave velocity within the medium  
284 depends on the elastic properties and the mass of the medium. Hence, it is possible to assess the medium elastic properties  
285 if its mass and the wave propagation velocity are known (Trtnik et al., 2009).

286 The ultrasonic pulse velocity method can be used in both laboratory (Popovics et al., 1990; Bogas et al., 2012) and on-  
287 site testing conditions (Lee et al., 2004). Main applications in the laboratory environment consist in the monitoring of  
288 material design characteristics and structural behaviour, e.g., strength development or deterioration in specimens subject  
289 to varying curing conditions or aggressive environments. Testing on the site allows to (i) measure the material uniformity  
290 (ii), detect cracking, (iii) have an estimation of the strength of the concrete, (iv) assess concrete deterioration (v) measure  
291 the thickness of concrete layers and (vi) to estimate the material elastic modulus.

292 Testing in the vicinity of reinforcement should be avoided as considerable uncertainty is introduced by the dominant  
293 influence of steel over concrete. The case of reinforcement parallel to the pulse path (Fig. 7) and transverse to the pulse  
294 path should be considered in order to apply corrections to the actual value of velocity of propagation in the concrete.  
295 Specifications on the use of the ultrasonic pulse velocity method are given in BS EN 12504 (BS EN 12504 2004) and  
296 ASTM C597 (ASTM C597 2016).

297



298

299 **Fig. 7** The ultrasonic pulse velocity method for reinforcing bars: case of reinforcement parallel to the pulse path

300

### 301 2.2.2. *Durability Tests*

302 Durability tests aim to assess the material characteristics affecting this particular feature as well as to identify the problem  
303 source. Deterioration of structural concrete may be due either to the action of chemical and weathering agents, and  
304 corrosion of embedded reinforcement steel is an element of major concern. Within this framework, durability tests involve  
305 different types of assessment such as the corrosion detection, moisture measurements and the absorption and permeability  
306 tests.

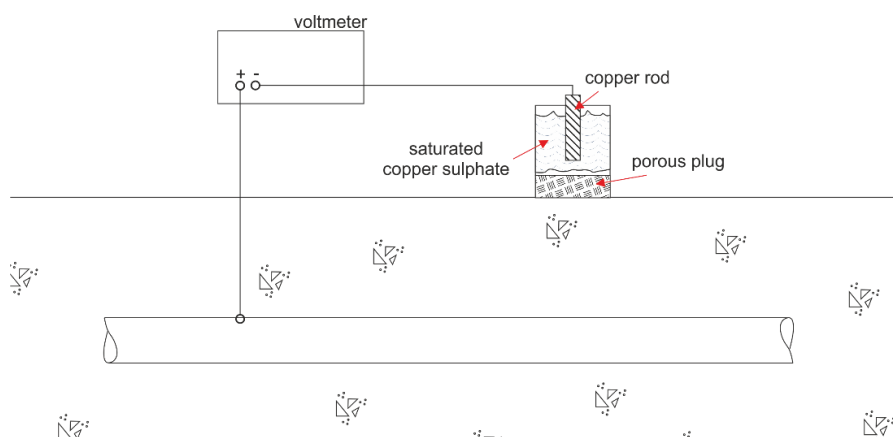
307

### 308 *Corrosion of Reinforcement*

309 Corrosion of reinforcement bars is an electro-chemical process that requires the contribution of moisture and oxygen.  
310 This process is often acknowledged as the major cause of decay in concrete structures. This occurrence may lead to a  
311 structural weakness due to the loss of the steel cross-section, surface staining and cracking or spalling. In some instances,  
312 internal delamination may also occur and, to this effect, EM methods are nowadays the most used for purpose. In this  
313 regard, battery-operated devices known as covermeters are employed (**BS 1881-204, 1988**). The fundamental working  
314 principle relies on the fact that steel affects the field of an electromagnet (i.e., an iron-cored inductor). An alternating  
315 current is passed through one of the coils, while the current induced in the others is amplified and measured. The reliability  
316 of this technique depends on factors affecting the magnetic field within the metre range. Among these, the presence of  
317 multiple reinforcing bars, laps or transverse steel, metal tie wires, variations in the iron content of the cement and use of  
318 aggregates with magnetic properties are worthy of mention.

319 More recently, the half-cell or rest-potential measurement method has been applied to assess areas of potential  
320 reinforcement corrosion. The method measures the potential of embedded reinforcing steel relative to a reference half-  
321 cell placed on the concrete surface (Fig. 8) (ASTM C876, 2009). It is also worth to mention that the half-cell potential  
322 method can be used in combination with resistivity measurements. The resistivity method works upon the ability of  
323 corrosion currents to flow through the concrete that can be assessed in terms of the electrolytic resistivity of the material.

324



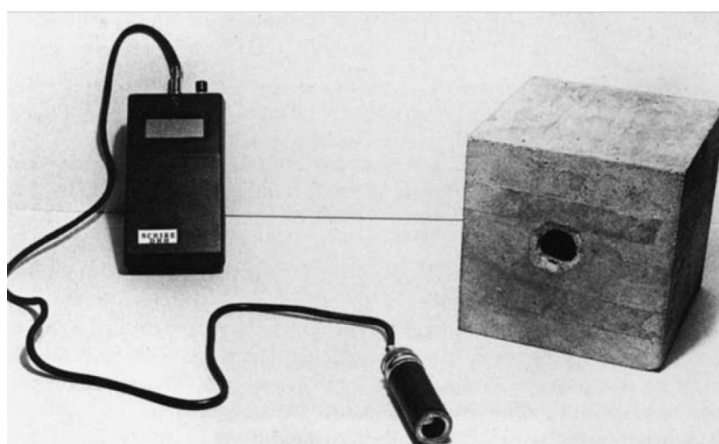
325

326 **Fig. 8** Measurement framework for the reinforcement potential method

327

### 328 *Moisture Detection*

329 Electrical methods (Fig. 9) are the most widespread within this assessment area as electrical resistivity measurements are  
330 highly sensitive to moisture. The dielectric properties of concrete are highly affected by the content of water, and factors  
331 such as the dielectric permittivity of the material and the dissipation factor can provide robust indications on moisture  
332 presence. It is known that the properties of a capacitor formed by two parallel conductive plates depend upon the  
333 characteristics of the separating medium. To this effect, the ratio between capacitances of the same plates when separated  
334 by the tested medium and by a vacuum identifies the dielectric permittivity.



335

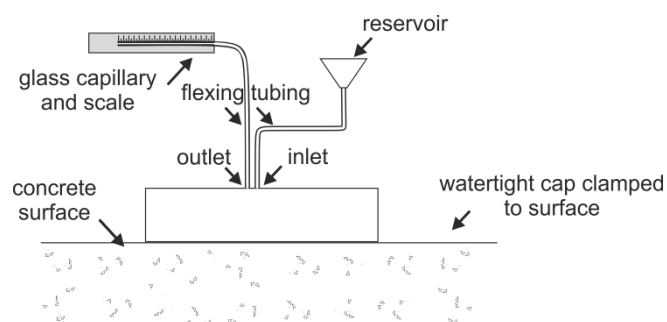
336 **Fig. 9** A humidity meter for detection of moisture (Bungey et al., 2006)

337

### 338 *Absorption and Permeability Tests*

339 The absorption and permeability tests are relevant for the assessment of concrete in water retaining structures or watertight  
340 basements, as well as to have an estimation of the material durability. Among the existing methods within this assessment  
341 area, the most utilised is the initial surface absorption method (Fig. 10) (BS 1881-208, 1996). Levitt (1969) has discussed  
342 the theory and the application of this technique and further analytical studies (Wilson et al., 1998) present the working  
343 mechanisms in detail.

344



345

346 **Fig. 10** The initial surface absorption test

347

### 348 2.2.3. *Performance and Integrity Tests*

#### 349 *Infrared Thermography*

350 The infrared thermography test is based on the analysis of images collected using a special camera (Fig. 11) during the  
351 cooling of a heated structure. Use of this technique has many potential advantages over physical methods for the detection  
352 of structural delamination, as it was discussed by Manning and Holt (1980). The detection of laminations or voids by  
353 infrared thermography is based on the difference in surface temperature between sound and unsound concrete under  
354 certain atmospheric conditions. First unsuccessful attempts were performed with a camera held by an operator standing  
355 on a bridge deck. The limited field of view and the oblique alignment made this method as impracticable. Therefore, the  
356 technique was improved using an elevated mobile platform to scan the deck from a height of up to 20 m, provided that  
357 the surface temperature differentials were greater than 2°C.

358 Procedures for use of infrared thermography in the investigation of bridge deck delamination are given in ASTM D4788-  
359 03 (ASTM D4788-03, 2013). Other applications to concrete structures involve the assessment of concrete moisture  
360 conditions influencing thermal gradients, and the location of hidden voids, ducts and services. The infrared thermography  
361 technique has nowadays reached a level of sensitivity to temperature differences of  $\pm 0.1$  °C and allows high-definition  
362 imaging and accurate temperature measurement.

363



364

365 **Fig. 11** Infrared thermography apparatus (ACI 228.2R-13, 2013)

366

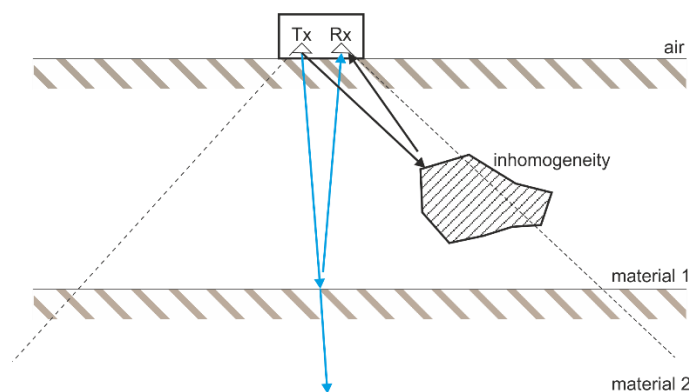
#### 367 *Ground-Penetrating Radar*

368 The use of GPR systems for the investigation of concrete structures has increased over the last 20 years (Cantor, 1984).  
369 GPR working principles rely on the propagation of EM waves through the material, with the speed and attenuation of the  
370 signal being affected by its electrical properties. Since radar wave reflections occur at the interface between materials



371 with different dielectric properties, the back-reflected signal can be interpreted to provide an evaluation of the properties  
 372 and the geometry of the subsurface. Within this context, a typical frequency range for applications to concrete is 500  
 373 MHz–2.5 GHz.

374 Three different approaches can be listed in regard to the use of GPR systems for the investigation of concrete structures:  
 375 (1) frequency modulation with continuous frequency of the transmitted radar within a default frequency range; (2)  
 376 synthetic pulses with varied frequency of the transmitted radar signal over a series of discontinuous steps; (3) impulse  
 377 radar-based systems working with a series of discrete sinusoidal pulses within a specified broad-frequency band and a  
 378 signal repetition rate of 50 kHz. A typical radar equipment configuration consists of an antenna in contact with the  
 379 concrete surface to produce a diverging beam of radiation. An alternative to this is to use a focused beam horn antenna  
 380 with an air gap of about 300 mm between the horn and the concrete surface. This latter technique is well-established and  
 381 operational details are provided in ASTM D4748-15 (ASTM D4748, 2015). An overview of the equipment, the field  
 382 procedures and the interpretation methods for the assessment of subsurface materials using GPR are provided in the  
 383 ASTM D6432-11 standard (ASTM D6432-11, 2011). More specifically, on concrete, the ASTM D6087-08 standard  
 384 (ASTM D6087-08, 2015) covers in detail a number of GPR procedures for the assessment of the concrete bridge deck  
 385 overlaid conditions with asphaltic concrete wearing surfaces. Guidance is provided for identifying the concrete or rebar  
 386 deterioration at or above the level of the top layer of reinforcing bars using GPR systems equipped with air-launched horn  
 387 antennas or ground-coupled antennas of central frequencies of 1 GHz and greater.



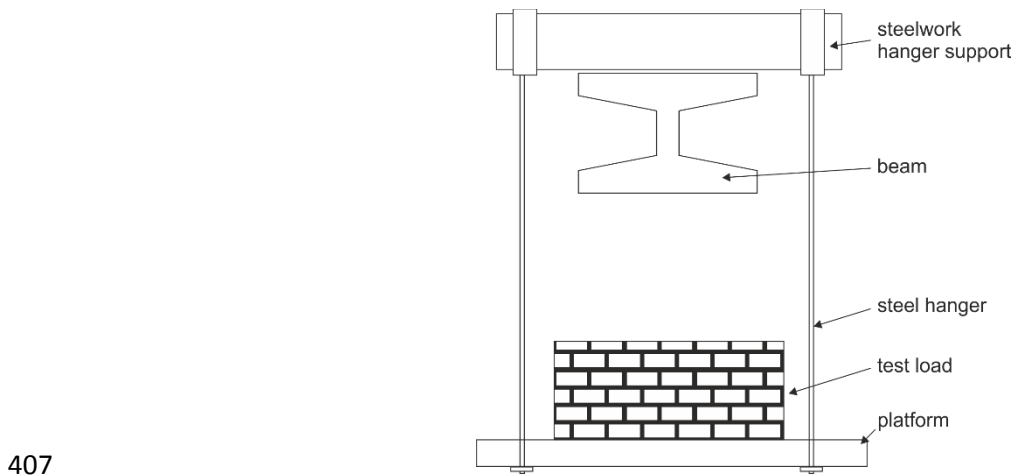
388  
 389 **Fig. 12** Investigation of a subsurface anomaly using a GPR ground-coupled antenna system

390

#### 391 **2.2.4. Load Testing**

392 Load testing may be necessary in case the strength of structural members cannot be adequately assessed using other  
 393 available tests. Two classes of load tests can be primarily identified, i.e., the static tests (most common) and the dynamic  
 394 tests, where variable loading are predominantly applied. An additional classification sorts load tests into non-destructive,  
 395 generally carried out in-situ, and destructive, generally applied on members removed from a structure.

396 The principal aim of these tests is to prove satisfactory structural performance under the effects of an overload above the  
 397 design working value. This is usually tested by measuring deflections sustained for a certain period of time under the  
 398 action of the overload. In-situ load tests require a minimum age of the concrete of 56 days in order to be carried out.  
 399 According to BS 8110 (BS 8110, 2003), loads must be added and removed incrementally, with the test load being applied  
 400 twice minimum at a time interval of 1 hour between two consecutive ones. An example of load test for light roof purlins  
 401 is shown in Fig. 13.  
 402 Reliability of in-situ load tests depends upon a number of key factors, such as a suitable preparatory work to avoid  
 403 potential restraints, the accuracy provided in the application of the load, the provision of an accurate datum for deflection  
 404 measurement and a careful allowance for temperature effects. Compliance of these requirements provides a reliable  
 405 indication of the behaviour of the member or the structure under test for short-term tests.  
 406



408 **Fig. 13** The load test for light roof purlins

### 410 3. THE GPR TECHNOLOGY

#### 411 3.1. Theoretical Background

412 GPR principles feed into the EM theory where the physics of the EM field propagation is described by the Maxwell's  
 413 equations and material properties are quantified by constitutive relationships. As a combination of these two factors, the  
 414 GPR signal is the output that provides information on the subsurface properties and configuration.  
 415 The propagation of the EM waves depends on the three main EM properties of the host material (Benedetto et al., 2017),  
 416 i.e., the dielectric permittivity  $\epsilon$ , the electric conductivity  $\sigma$  and the magnetic permeability  $\mu$ .  
 417 The dielectric permittivity and the electric conductivity are strictly related to the EM wave features.  $\epsilon$  affects the wave  
 418 velocity, and  $\sigma$  controls the wave attenuation. On the contrary, the magnetic permeability  $\mu$  does not relate with the  
 419 propagation of the wave for all the non-magnetic materials, as it is equal to the free-space magnetic permeability  $\mu_0$ .

420 On the other hand, the main factors affecting the penetration depth are the frequency of the emitted signal (for structural  
421 inspections, antennas with central frequencies above 1 GHz are used) and the type of material investigated.

422 From a theoretical point of view, the physics of EM fields is described by the Maxwell's equations as follows:

$$423 \quad \nabla \times \vec{E} = -\frac{\partial (\vec{B})}{\partial t} \quad (1)$$

$$424 \quad \nabla \times \vec{H} = \vec{J} + \frac{\partial (\vec{D})}{\partial t} \quad (2)$$

$$425 \quad \nabla \cdot \vec{D} = q \quad (3)$$

$$426 \quad \nabla \cdot \vec{B} = 0 \quad (4)$$

427 where:

428  $\vec{E}$ = strength vector of the electric field (V m<sup>-1</sup>);  $q$ = electric charge density (C m<sup>-3</sup>);  $\vec{B}$ = density vector of the magnetic flux (T);  $\vec{J}$ =  
429 density vector of the electric current (A m<sup>-2</sup>);  $\vec{D}$ = electric displacement vector (C m<sup>-2</sup>);  $t$ = time (s);  $\vec{H}$ = intensity vector of the magnetic  
430 field (A m<sup>-1</sup>).

431  
432 Material properties are instead quantified by the following constitutive relationships:

$$433 \quad \vec{J} = \sigma \vec{E} \quad (5)$$

$$434 \quad \vec{D} = \varepsilon \vec{E} \quad (6)$$

$$435 \quad \vec{B} = \mu \vec{H} \quad (7)$$

436 Combination of the EM fields' theory and the material properties allows to describe comprehensively a GPR signal.

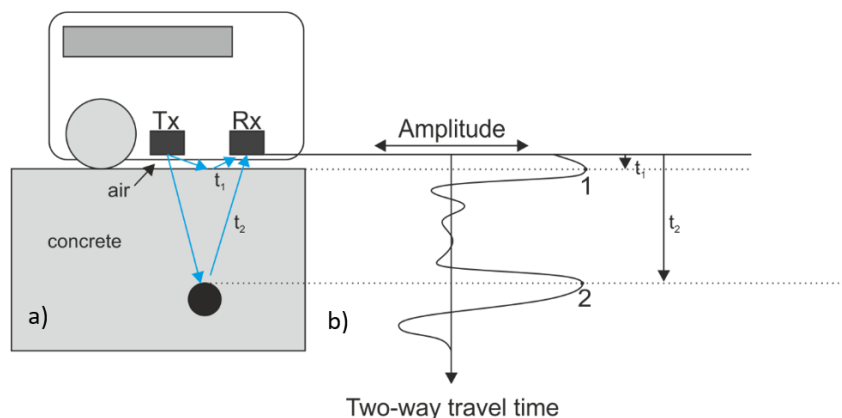
437

## 438 **3.2. Working Principles**

### 439 **3.2.1. Overview**

440 GPR is a geophysical inspection technique used for the non-destructive investigation of the subsurface. Fig. 14 represents  
441 a GPR output from a survey layout typical of a reinforced structure. An EM wave is emitted towards the surface by a  
442 radar with a fixed central frequency using one or more antennas. The signal is then received as a function of the material  
443 properties and characteristics of the receiving antenna. A GPR signal is characterised by a series of peaks with amplitude  
444 values dependent on three main factors, i.e. the nature of the reflector, the nature of the travelled medium and the curve  
445 of the applied amplification (Noon, 1995). Typically, three visualisation modes can be listed for a GPR signal that provide  
446 three different levels of information: i) an A-scan, i.e. a single radar trace along the depth axis; ii) a B-scan, i.e. a set of  
447 sequential single radar traces collected along a specific scanning direction; and iii) a C-scan, i.e. a set of B-scans  
448 extrapolated at a certain spacing along the depth axis (Benedetto et al., 2017).

449 Various information can be collected with GPR, such as the two-way travel time distance between reflection peaks at  
 450 layer interfaces/target positions (e.g. rebars), the amplitude and the phase of a signal. To this effect, locating rebars and  
 451 estimating cover thickness are among the major applications of GPR to reinforced concrete structures.



452  
 453 **Fig. 14 a** Layout of a survey on a reinforced concrete structure using a ground-coupled GPR system, **b** corresponding  
 454 GPR signal output

### 456 3.2.2. Configuration of the Antennas

457 A GPR system is configured as mono-static when a unique antenna operates as both transmitter and receiver, and bi-static  
 458 in case of separated transmitter and receiver. In addition, GPR systems can be sorted as ground-coupled or air-coupled,  
 459 as a function of the type of antennas. Ground-coupled antenna systems work in direct contact with the surface and allow  
 460 for a higher penetration depth. A typical frequency range for these systems is from 80 MHz to 2000 MHz. Air-coupled  
 461 antennas work attached to an inspection vehicle and allow for a faster acquisition. The radar apparatus are usually set up  
 462 at 0.15 m÷0.50 m above the surface and the central frequency ranges typically from 1000 MHz to 2000 MHz.  
 463 For concrete structure inspections, ground-coupled antennas with central frequencies above 1 GHz are usually employed  
 464 to provide a trade-off between accuracy and depth of inspection (Hugenschmidt, 2002; Burgey, 2004).

### 466 3.3. GPR Applications in Reinforced Concrete Materials and Structures

467 GPR is a fundamental method for quality control and routine inspections of reinforced concrete in civil engineering  
 468 infrastructures (Wiwatrojanagul et al., 2017). The suitability of GPR is related to the presence of concrete (low-conductive  
 469 material) and rebars (high-conductive materials), which allow for effective detection in the composite material. The very  
 470 first application of a radar system in detecting metal objects dates back to 1904 (Ulricksen, 1982), whereas the use of  
 471 GPR in structures started to gain momentum in 1980s (Forde, 2004). Nowadays, GPR is used for many applications in

472 the area of reinforced concrete materials and structures, although the location of reinforcing bars is one of the most  
473 widespread (Pucinotti and De Lorenzo, 2006).

474 The main applications in the literature within this subject area are: i) the location of reinforcing bars, ii) the detection of  
475 rebar corrosion, iii) the estimation of the rebar size, iv) the evaluation of the concrete cover, v) the structural detailing of  
476 anchorages and joints in major structures/infrastructures, and v) the assessment of concrete characteristics (i.e. uniformity,  
477 voids, cracks, strength and durability) by analysing the material dielectric properties.

478 In regard to the assessment of reinforcing bars, damage to these elements is the most serious source of structural decay in  
479 concrete structures, with many economic implications leading to repair or, eventually, replacement of the main structural  
480 element.

481 Locating reinforcing bars is of primary importance in both quality control inspections of new structures and infrastructures  
482 as well as in the structural detailing of existing ones (Benedetto et al., 2012; Stryk et al., 2013; Stryk et al. 2018). In  
483 addition, it is known that corrosion of rebars, usually due to moisture ingress and chloride ion exposure (Shi et al. 2012),  
484 can cause subsurface cracking and spalling of the concrete cover. This occurrence promotes water and chloride  
485 infiltration, which in turn can generate delamination around the reinforcing bars. Within this framework, the GPR  
486 technique has proven its viability to detect corrosion in rebars due to signal attenuation effects from moisture and chloride  
487 ingress (Laurens et al., 2000; Kim et al., 2003; Hugenschmidt and Loser, 2008). Estimating the size of rebars is another  
488 topic of major interest. Although several promising approaches and numerical-based applications have been reported in  
489 the literature, further research is still required in order to make GPR a reliable and established technique for the purpose  
490 (Utsi and Utsi, 2004; Zanzi, 2012; Giannakis et al., 2019). GPR has been extensively used for the assessment of concrete  
491 covers as both a self-reliant equipment (Klysz et al., 2004; Al-Qadi and Lahouar, 2005; Hasan and Yazdani, 2014) and in  
492 combination with other NDT methods (Dèrobert et al., 2008).

493 The assessment and the health monitoring of major reinforced-concrete structures and infrastructures is another important  
494 application area for the GPR techniques. To this effect, an early diagnosis of decay at key areas, such as anchorages in  
495 building foundations and pre-stressed beams, joints in bridge decks (Benedetto et al., 2012) and tunnel linings (Alani and  
496 Tosti, 2018), can limit the risk of known consequences such as accidents, closures and ultimately devastation. The  
497 assessment of concrete characteristics is another research area of major development. In this regard, the detection of  
498 cracks (Benedetto, 2013) and voids (Cassidy et al., 2011) as well as the assessment of concrete strength and durability  
499 features (Sbartai et al., 2009; Villain et al., 2010) have been widely investigated in the literature.

500

501 **4. RESEARCH METHODS ON THE USE OF GPR IN REINFORCED CONCRETE STRUCTURES: AN**  
502 **HISTORICAL REVIEW**

503 An historical review of research within the context of the applications of GPR to reinforced concrete structures is given  
504 in this Section. Discussed research methods are sorted according to the scale of investigation (i.e. laboratory and numerical  
505 environments, and real-life scales of investigation) and the main application areas identified above. A chronological  
506 overview of the main research findings achieved within every application area is therefore reported. Overall, it was  
507 observed that not all the application areas have been covered at both the laboratory and the real-life scale of investigation.  
508 A lack of case studies has been noticed in certain areas in the case of applications to real-life scenarios. This could be due  
509 to practical constraints preventing full control of the boundary conditions, as well as to clear difficulties in validating  
510 research outcomes by a destructive way of practice. On the contrary, these factors become easier to control and analyse  
511 in a laboratory environment. It was also noticed that the use of pulsed GPR systems was preferable to the use of stepped-  
512 frequency continuous-wave (SFCW) radar systems. This is likely due to the higher cost of electronics, an overall major  
513 complexity in data processing (i.e. high computational requirements) and a lack of dedicated commercial software  
514 (Gagarin and Mekemson, 2016).

515

#### 516 **4.1. Laboratory-scale Investigations and Numerical Developments**

##### 517 **4.1.1. Location of Reinforcing Bars**

518 The location of reinforced bars has had more applications in real-life scenarios rather than in a laboratory environment.  
519 This may be related to the fact that this operation is relatively practical, as it requires the identification of the apex of the  
520 hyperbolic target reflections. Migration processing can help to estimate the actual propagation velocity of the EM wave  
521 through the concrete cover and, therefore, to precisely locate the rebar position.

522 Extensive research was carried out by Bungey et al. (1993). Tests were developed in a laboratory to identify the position  
523 of reinforcing bars using a 1 GHz GPR antenna system. The effects of round steel reinforcing bars with diameters between  
524 6 and 32 mm have been analysed at depths of up to 280 mm below the surface of the concrete. This has been facilitated  
525 by the use of a timber-analogue simulation tank in which the dielectric and conductivity properties of the concrete were  
526 replicated using an oil-water emulsion. In excess of 250 different configurations were analysed and results were validated  
527 by way of comparison with a number of concrete samples and by site trials on members of known construction details.  
528 More recently, Zhou et al. (2018) used GPR in combination with the electromagnetic induction (EMI) method to  
529 accurately locate reinforcing bars. The authors utilised a number of four casted concrete specimens with 11 embedded  
530 steel rebars, proving a correct estimation of the rebar size for 10 out of the 11 rebars.

531 In regard to the use of numerical modelling for rebar location, Alvarez et al. (2017) used the gprMax numerical simulator  
532 tool (Giannopoulos, 2005) to generate realistic, big data sets of different concrete covers for training-driven machine  
533 learning approaches. Although the approach was tested against a concrete-analogue scenario made of a sand-based

mixture, the authors found the simulation package to be suitable for generating realistic outputs. Lachowicz and Rucka (2018) presented numerical and experimental investigations to localise reinforcing bars using GPR methodology. In more detail, a new approach for the numerical modelling of GPR in complex reinforced concrete structures with the use of a 3-D finite-difference time-domain (FDTD) model was presented. Comparison between simulated results and real scans has proven the viability of the proposed model to work on a complex reinforcement system. More recently, Kien et al. (2018) presented an automated rebar-picking algorithm for GPR data of concrete bridge decks with the aim of identifying and locating rebars. The algorithm is based on the Limited and Simplified Hyperbolic Summation (LSHS) technique where the width of migration is limited and a counter is used to check if a hyperbolic signature exists in a sub-region of the GPR image. The application of the algorithm to GPR data was promising and in a good agreement with the existing techniques based on manual rebar picking.

544

#### 545 **4.1.2. Detection of Rebar Corrosion**

546 The interest of using GPR for detection of steel bar corrosion is growing nowadays. An early diagnosis can support  
547 decision-makers to prevent the spreading of damage throughout the structure and to containing maintenance costs. To  
548 this effect, research efforts have been directed towards the identification of wet areas nearby the rebars, where the  
549 corrosion process is most likely to be found. It was observed that most of the research methods developed for this  
550 particular application area have been performed in a laboratory environment. This is due to more favourable conditions  
551 in controlling the ingress of corrosion agents and, hence, to identify areas of potential decay. It was also noticed that  
552 results obtained from a number of these research studies were implemented on the site at a later stage and combined with  
553 conventional methods.

554 Narayanan et al. (1998) analysed the statistical variance of rebar reflectivity to detect corrosion using 400 MHz and 900  
555 MHz antenna frequencies. Results have proven a better agreement of the 900 MHz central frequency with the ground-  
556 truth data. A subsequent research was carried out to identify a threshold between healthy and corroded rebars (Narayanan  
557 et al., 2003). The variance simulations supported to identify the reflectivity of the corroded rebar. Laurens et al. (2000)  
558 and Barnes et al. (2008) observed that lower relative reflection amplitudes and larger travel times of the GPR signal are  
559 an indication of a high corrosion and deterioration of rebars. In this regard, it was observed that moisture and chlorides  
560 dissolved within the concrete attenuate the signal back-reflected from the rebars. This occurrence decreases the wave  
561 propagation velocity and increases the arrival times of reflection peaks. A relationship between the amplitude data of the  
562 reflections from rebars and the corrosion process was confirmed by Hubbard et al. (2003). The authors used GPR and  
563 electrical impedance to analyse rebar corrosion in a reinforced concrete block. Research outcomes were validated by  
564 visual examination of the reinforcing bar corrosion state via destructive analysis of the experimental block. Results have

565 proven the viability of GPR in providing indication of alterations at the interface of the reinforcing bar surface and the  
566 surrounding concrete. GPR was also proven to provide a higher spatial resolution than the electrical impedance. These  
567 outcomes were valuable indications that GPR can be used for an early detection of corrosion-related processes of decay.  
568 More recently, Raju et al (2018) employed a hand-held ground-coupled GPR system equipped with an antenna frequency  
569 of 2.6 GHz in order to assess reinforcing bar corrosion. A saltwater solution was used to control the corrosion process in  
570 a laboratory environment. The authors related the corrosion with the maximum reflected amplitude obtained from the  
571 GPR scans. Results have proven that the reinforcing bar corrosion mass loss was higher for longer corrosion periods,  
572 larger reinforcing bar size, and shorter covers. It was also noticed that the GPR amplitude values increased proportionally  
573 with the corrosion activity.

574 Few research is reported on the use of the numerical simulation for detection of corroded areas. This is due to the  
575 complexity in the physics and the chemistry of the process as well as to the effect of the corrosion on the surrounding  
576 construction elements. This complex scenario involves multiple scatterings of the signal from the rebar-concrete system  
577 and the corroded areas in rebars and/or other defects in the concrete. Bachiri et al. (2018) developed a numerical model  
578 of a bridge deck and analysed scenarios of healthy and corroded rebars. Results have shown that a proper selection of the  
579 working frequency value is crucial to increase the survey resolution and the likelihood to detect corrosion. The authors  
580 claimed that an experimental validation is however required to confirm the outcomes of the simulation.

581

#### 582 **4.1.3. Estimation of Rebar Size**

583 Estimation of rebar size is a topical and very challenging application area, and research is still in progress to provide  
584 viable and comprehensive methodologies. In this regard, it was observed that all the methods have been developed in a  
585 laboratory environment, proving that the research applicability in this area is at an early stage of development. A common  
586 approach to this problem is to use numerical modelling and to validate the observations with experimental activities  
587 carried out on samples in a laboratory environment.

588 Utsi and Utsi (2004) proposed a three-dimensional numerical model assuming one metal bar for a range of diameters and  
589 cover depths. Frequencies of 2 GHz and 4 GHz were used for simulation purposes. Results showed that the amplitudes  
590 of the signals back-reflected from the rebars are proportional to their size and the frequency is inversely proportional to  
591 the depth of the investigation. These results were compared with measurements of rebars placed at known distances below  
592 a 4 GHz GPR system with free space in between. According to the authors, an accuracy of about 20% can be reached in  
593 estimating rebar size. Another issue was encountered in case of a rebar spacing lower than the wavelength of the used  
594 GPR, as this may compromise the data collection for rebar size estimation purposes.



595 Chang et al. (2009) reported a physical model based on the use of digital image GPR for measuring the radius of  
596 reinforcing steel bars in concrete. To this purpose, several specimen of  $120 \times 60$  cm dimensions were produced using  
597 rebars of 1 cm and 1.6 cm radii. Concrete cover depths varied from 4 to 10 cm. GPR radargrams were subject to a series  
598 of digital image processing stages, followed by different power reflectivity within the energy zone during the motion of  
599 the GPR antenna along the reinforced concrete surface. Power reflectivity for vertically-oriented migration traces was  
600 generated. The authors argued that the distance between a variant power reflectivity and the long dimension radius of an  
601 energy footprint must be considered for the evaluation of a rebar radius. Results indicate an error for the estimation  
602 comprised within 7%. Zanzi and Arosio (2012) used a 1 GHz and a 2 GHz GPR systems on two concrete specimen of 40  
603 cm width, heights of 15 and 18 cm, and lengths of 105 and 110 cm, respectively. Rebar diameters from 6 to 40 mm,  
604 spaced 20 cm each other were analysed. The experimental scenario was replicated using numerical simulations. The  
605 authors identified the interference of the background signal with the wavelets scattered by the rebars as the main issue to  
606 address in order to achieve quality information. Near-field and bandwidth-related issues seemed to be of less priority in  
607 generating potential misfits between theoretical and real measurements. A set of three antennas with nominal frequencies  
608 in the 600-MHz  $\div$  2000 MHz range were indicated as a good trade-off for detection of rebars with diameters up to 40  
609 mm. However, a cross-check of the results in a real-life scenario was suggested by the authors in order to consider the  
610 impact of the concrete cover and the quality upon the reliability of the rebar size estimation. To this effect, use of a high-  
611 frequency dual-polarised antenna system was suggested along with a survey methodology aimed at collecting a robust  
612 statistical population of rebar points.

613 In view of the high density distribution of rebars in a reinforced concrete structure, finding comprehensive and fast  
614 inspection algorithms and methodologies for detection of rebar corrosion has been identified as a major research subject  
615 area. In this regard, Shaw et al. (2003) developed a neural network approach to automate the estimation of the rebar size  
616 diameter from data collected with the transducer axis parallel and orthogonal to the bar. To this effect, an emulsion  
617 analogue tank simulating the properties of concrete, was used to produce training data and the resulting neural network  
618 was then tested on actual reinforced concrete slabs. Results have proven a good estimate of the rebar size for both high  
619 and low concrete covers. More recently, Mechbal and Khamlichi (2017), proposed a method for the estimation of the  
620 rebar radius by data processing of B-scan radargrams collected orthogonally to the rebar axis. The approach relies on use  
621 of the hyperbola fitting method and the information from diffracted amplitudes. Hyperbola fitting has proven to be  
622 extremely sensitive to noise affecting raw data points. It was also demonstrated that use of extra information in terms of  
623 maximum diffracted amplitudes enhanced the accuracy of rebar radius estimation.

624 Giannakis et al. (2019) proposed a near-real-time forward modelling approach for GPR based on a machine learning (ML)  
625 architecture. The method was used to determine location and size of reinforcement bars in concrete. The authors have

used simulations to train a ML-based forward solver and validated the approach with real data collected on a concrete specimen. A 1.5-GHz antenna frequency was used to investigate a rebar size range between 4 and 50 mm and a cover depth varying from 0 to 300 mm. Results have proven a good level of accuracy in the estimation of the target parameters, including a maximum error in the estimation of the rebar size of 6 mm. The authors emphasised on the viability of using a full-waveform inversion approach in near-real-time computation conditions for application to more complex scenarios.

#### **4.1.4. The Evaluation of the Concrete Cover**

The evaluation of the concrete cover has found more applications in real-life scenarios rather than in a laboratory environment. Within this framework, major laboratory-based research was carried out by Dérobert et al. (2008) where capacitive and GPR techniques were used to evaluate the concrete cover. A number of 10 concrete slabs with dimensions of  $60 \times 60 \times 12$  cm and compressive strengths of 25 and 40 MPa were tested using a 1.5-GHz ground-coupled antenna system. The authors emphasised the importance of using the capacitive technique to provide more accurate results for the concrete cover, prior to any extensive GPR investigation. Use of a higher frequency of investigation was also suggested in order to allow for more effective measurements across the rebar offsets compatible with standard mesh grids. More recently, Zhou et al. (2018) have proven that integration of GPR with other complementary NDT methods can be a viable approach to provide more accurate information. The authors used the GPR and the EMI techniques with a maximum estimation error for the cover thickness of 6.7%. On the other hand, a difficulty in providing an effective measurement and an estimation of rebars in a densely-meshed net was identified as a limitation of the proposed approach.

#### **4.1.5. The Assessment of Concrete Characteristics by Dielectric Properties**

Several research studies are discussed in the literature in regard to the assessment of concrete characteristics by analysis of its dielectric properties. It is known that among the factors affecting the dielectric properties of a concrete material (Knoll, 1996), water is a primary and dominant parameter affecting the relative dielectric permittivity (Soutsos et al., 2001). According to Lai et al. (2009), other factors influencing the dielectric properties of concrete are: i) the EM frequency; ii) the water-to-cement ratio; iii) the porosity; iv) the ions in pore solution and v) the clay minerals with a wide range of porosity values and specific surfaces. Factors such as the cement type, the salt impregnation, the presence of pulverised fuel ash and ground granulated blast furnace slag, and the temperature during tests were reported to be of minor or negligible significance. Similarly, the effects of different types of dense aggregates on the relative dielectric permittivity were accounted to be negligible (Soutsos et al., 2001).

Louzli et al., (2002) developed a method to determine the complex dielectric permittivity of concrete mixes and to assess concrete voids. A number of five bare concrete slabs and nine slabs with a simulated delamination (simulated by placing

657 polystyrene plastic pieces inside the slabs) were constructed. All slabs were cast at 1.5 x 1.5 x 0.127 m and five different  
658 concrete mixes were used. Effects of delamination on the signal were analysed by way of comparison between the signals  
659 from the bare concrete slabs and the signals from the slabs with simulated delamination. In this regard, modelling the  
660 reflected signals with an average complex dielectric permittivity over the full GPR frequency range was found to provide  
661 comparable outputs in terms of synthetic and real (measured) waveforms. In addition, presence of voids located at 50 mm  
662 from the surface overlap with the surface reflection were observed to distort the shape of the reflected signal.

663 Several studies have investigated the EM behaviour of wet concrete. Laurens et al. (2005) analysed the effect of moisture  
664 on the radar waves propagating through concrete slabs. To this purpose, a GPR system equipped with a 1.5 GHz ground-  
665 coupled antenna was used to collect data at different degrees of saturation in homogeneous moisture distribution  
666 conditions. An investigation into the amplitudes, the velocities and the frequency spectra of the collected waveforms was  
667 developed to assess the GPR capability in estimating concrete moisture. The authors found the moisture to affect the  
668 transmitter-receiver direct wave.

669 GPR was also reported to characterise pore systems of air/water cured concrete through injection of continuous water  
670 over a period of 48 hours (Lai and Tsang, 2008). To this purpose, a 1 GHz radar system was used and the real part of the  
671 complex permittivity and the energy attenuation were measured. As an outcome of the investigation, it was also  
672 emphasised a good potential of GPR in identifying the progress rate of the water front in concrete walls or slabs.

673 Sbartaï et al. (2009a) applied a combined analysis of radar technology and artificial neural networks for predicting water  
674 and chloride content of concrete. Four different concrete mixes were used to produce a number of 72 concrete slabs with  
675 dimensions of  $25 \times 25 \times 8$  cm. Moisture and chloride variations ranging from 0 to 16.5% and from 0 to  $6.5 \text{ kg/m}^3$  were  
676 analysed, respectively. Input data to the neural networks were extracted from time domain signals of direct and back-  
677 reflected radar waves. Results highlighted the viability of using optimised statistical models to predict water and chloride  
678 content of laboratory slabs with maximum absolute errors of about 2% and  $0.5 \text{ kg/m}^3$ , respectively.

679 Sbartaï et al. (2009b) assessed water content in concrete samples at several degrees of saturation using a frequency  
680 analysis of the direct wave signal collected from a 1.5 GHz ground-coupled antenna system. A Fast Fourier Transform  
681 algorithm was used to extract the frequency spectra of each signal and, based on the experimental results, an empirical  
682 model was proposed to relate the frequency attenuation and the concrete water content. By way of comparison between  
683 the results achieved in the time and in the frequency domains, the frequency-attenuation analysis was found to be very  
684 effective for concrete moisture assessment purposes.

685 Villain et al. (2010) used GPR and capacitive probes to test a number of 81 slabs made of 9 different concrete mixes.  
686 Concrete porosity ranged between 12.5 and 18% and five different water contents were reproduced. A cross-correlation  
687 model was developed and durability-related parameters such as the porosity, the water content and the chloride were

688 successfully predicted. The authors also observed that the viability of the proposed methodology is maximum if the cross-  
689 correlation model is calibrated in laboratory using core samples from a real-life concrete structure.

690 Kalogeropoulos et al. (2013) proposed a multi-stage approach for the calculation of the EM parameters in concrete slabs.  
691 A GPR dataset was collected using 1.2 GHz horn antennas on concrete specimens with different chloride gradients. A  
692 first approach was developed to estimate the relative dielectric permittivity  $\epsilon_r$  and the conductivity  $\sigma$  of the concrete.  
693 Results demonstrated an accuracy of  $\pm 0.15$  for  $\epsilon_r$  and  $\pm 10$  mS/m for  $\sigma$ . A second approach, based on the use of an extended  
694 full-waveform inversion forward model, enabled the creation of conductivity gradients for multi-layered media in  
695 agreement with literature outcomes.

696

## 697 **4.2. Real-life Investigations**

### 698 **4.2.1. *Location of Reinforcing Bars***

699 Many applications of GPR exist in real-life scenarios in regard to the location of reinforcing bars and the use of GPR  
700 within this specific application area was already reported by Ulriksen (1982) a few decades ago.

701 According to Hamasaki et al. (2003), the working principles of EM-based methods allow for the effective detection of a  
702 wave reflected back by a reinforced bar. This is mostly due to the different dielectric properties of the two materials  
703 involved. The authors argued that location, direction, and diameters of rebars can be determined based on the variations  
704 of the EM field.

705 Barrile and Pucinotti (2005) used GPR to identify the position of principal and secondary (i.e. longitudinal steel bars and  
706 stirrups) reinforcement bars located on the beams and columns of reinforced concrete structures. The aim of the research  
707 was to assess the seismic vulnerability of a 40-year-old building. The survey was carried out using a ground-coupled  
708 antenna system with a central frequency of 1.6 GHz. Longitudinal and transversal scanning were performed in order to  
709 obtain a dimensional relief of the rebar position. The authors obtained detailed information on the number, position and  
710 spacing of reinforcing bars. It was also possible to evaluate the thickness of the concrete covers. With a view to automation  
711 and reduction of data processing times for the location of reinforcement, Shaw et al. (2005) proposed an integrated method  
712 for the post-processing of GPR data using a neural network-based approach. The hyperbolic images from rebars were  
713 collected using a multi-layer perceptron (MLP) network with a single hidden layer containing 8 nodes. This allowed  
714 recognition of a more simplified hyperbolic shape and, hence, a lighter computational load for the algorithm. This  
715 approach has proven viability in automating the identification and location of embedded steel reinforcing bars using GPR.  
716 Soldovieri et al. (2006) presented an inverse scattering approach for the accurate location of rebars in reinforced concrete  
717 structures. The benefit of this method is that the quality of the final output is less dependent upon the expertise of human  
718 operators and, hence, less subject to interpretation errors.

719 Benedetto et al. (2012) reported the use of a multi-channel high-frequency array GPR system for the faster acquisition of  
720 data in bridges. The authors provided details of rebar location using differently-polarised antennas. Stryk et al. (2013)  
721 presented a comprehensive case study where positions of rebars were identified with a two-channel ground-coupled  
722 antenna system with a central frequency of 1.6 GHz. Rebar positions were checked against standard requirements. More  
723 recently, Seren and Saricicek (2017) carried out an on-site investigation for health monitoring purposes. The survey was  
724 developed using a 2.7 GHz antenna system and migration techniques were applied to provide a correct location of the  
725 reinforcing bars. Hyperbolic features were observed from 20 cm up to 1 m of depth. Stryk et al. (2017) reported a  
726 comparative analysis for assessing the accuracy of GPR systems in rebar location. A number of GPR from different  
727 manufacturers with antenna frequencies of 1.6 GHz were used for this purpose. The authors reported an accuracy of 10  
728 mm and 5 mm in horizontal and vertical directions, respectively. It was also recommended to use a two-channel device  
729 to measure the target location in two points within a single overrun and achieve additional information about the  
730 orientation of rebars. Miramini et al. (2018) assessed the structural health of a 45-year-old pedestrian bridge deck after  
731 the validation of the GPR technique on a strong concrete floor with known construction drawings. From the scanned  
732 results, the location and orientation of the reinforcing bars were established and their diameters were estimated. The  
733 signals demonstrated the absence of reinforcement corrosion or concrete damage in the form of delamination or cracking.  
734

#### 735 **4.2.2. Detection of Rebar Corrosion**

736 Research in this particular application area is not as numerous for real-life scenarios as for applications carried out in a  
737 laboratory environment. This is likely related to a larger difficulty encountered for the validation of research outcomes  
738 by a destructive way of practice. Within this framework, focus of research was observed to be primarily directed towards  
739 the development of: i) new algorithms capable to relate corrosion with specific signal features, and ii) automatic  
740 algorithms for fast detection of rebar corrosion.

741 Wang et al. (2011) proposed a method for detection of rebar mat signatures in concrete bridges from GPR data using  
742 partial differential equations. The aim of the research was to identify delamination within a real-life bridge deck. The  
743 proposed algorithm was tested using both synthetic and real GPR images. The experimental results have proven the  
744 algorithm accuracy and reliability, although weak image contrasts and low signal-to-noise ratios were found. A  
745 comparison between deterioration maps for the bridge deck generated by the proposed algorithm and a traditional manual  
746 method has proven the viability of the approach. The authors emphasised on the need to test the algorithm against a larger  
747 GPR image dataset.

748 Martino et al. (2014) developed a method to identify thresholds relating GPR signal amplitudes and rebar corrosion. The  
749 authors used GPR and the half-cell potential (HCP) method on one bridge deck removed from service, a number of 17

artificially-corroded slabs, and one in-service bridge deck. The HCP method was employed to measure the amount of active corrosion. A significant correlation between the two NDT methods was found for each case study. Receiver operating characteristic (ROC) curves were used to systematically set a threshold for the GPR and deteriorated areas of the deck were identified with an accuracy of over 87% for every scenario. Abouhamad et al (2017) proposed a technique based on image analysis for interpreting GPR data with the aim to create a systematic framework. Benefit of using this approach was reported to overcome potential drawbacks of the numerical analysis in interpreting surface anomalies as false alarms. The proposed systematic framework was supported by various GPR profiles showing several causes of signal attenuation and their interpretation in regard to deterioration or rebar corrosion conditions. The approach was validated on two real bridges by way of comparison between the outcomes from field visual inspections, camera images and concrete core sampling. The method has proven to be accurate in identifying corrosion-related and non-corrosion-related factors.

761

#### 762 **4.2.3. The Evaluation of the Concrete Cover**

The evaluation of the concrete cover has found major interest within the field for the assessment of the structural integrity of civil engineering infrastructures and several case studies have been reported in the literature about the use of GPR in real-life scenarios.

Al-Qadi and Lahouar (2005) proposed a GPR-based image-processing technique for the automatic detection of the rebar parabolic signature from rigid pavements with a high-frequency ground-coupled antenna system. The authors used the reflected parabolic shape to develop a theoretical reflection model with the aim of estimating the pavement dielectric permittivity and the rebar depth. The algorithms were validated on GPR data collected on a known continuously reinforced concrete pavement section. The technique has proven to be accurate and has showed an average error of 2.6% for the estimated cover depth of rebars. Alani et al. (2013) presented an application on a road bridge with the purpose of providing an accurate estimate of the concrete cover conditions. The authors observed a clear similarity between areas affected by the ingress of moisture and areas with damaged concrete covers. Dinh et al. (2016) reported a substantial piece of experimental research carried out on twenty-four bare concrete bridge decks using a ground-coupled 1.5-GHz GPR antenna system. Identified objectives were i) to comprehend the impact of rebar depth on the GPR signal loss, ii) to identify a method for providing an objective comparison of the GPR data from different bridge decks, and iii) to normalise depth-amplitude effects for ensuring a more consistent assessment of bridge decks. The authors observed that mechanisms governing depth-amplitude effects are mostly related to the effects of dielectric and conductive losses. However, the authors concluded that specific depth-amplitude functions must be created in relation to specific overlay types in bridge decks.

781

782 **4.2.4. Structural Detailing of Anchorages and Joints in Major Structures/Infrastructures**

783 Structural detailing using GPR for assessment of the integrity of reinforced concrete structures has been reported in the  
784 literature. A methodological overview on the use of GPR for surveying buildings, bridges and tunnels was given by  
785 Daniels (2004). According to the author, suitability of GPR in structural inspections is mostly related to the collection of  
786 reflection data from main structural boundaries. To this effect, the main boundaries to survey in a structure were identified  
787 to be located at the interface between: i) a solid material and a significant void; ii) good and poor quality concrete; iii)  
788 concrete and insulation materials; iv) large metal inclusions as reinforcement bars or other components; v) dry and wet  
789 materials.

790 It was observed that most of the research in this specific application area was carried out over the past decade. This is  
791 likely due to the fact that GPR is nowadays an established method in civil engineering and technological advances have  
792 fostered its use in many challenging inspection tasks.

793 Within this framework, Hugenschmidt et al. (2010) discussed on the viability of using a high-resolution multi-sensor and  
794 multi-polarised GPR system to investigate concrete retaining walls. The authors proposed two new processing methods,  
795 i.e., i) a full 3D processing followed by a data fusion approach and ii) an inverse scattering followed by a data fusion  
796 approach. The methods were compared with a standard processing protocol working in the two dimensions and they were  
797 tested on a retaining wall in Switzerland. Results have proven the viability of the two innovative approaches. Benedetto  
798 et al. (2012) used a multi-channel dual-polarised GPR system to monitor four reinforced concrete bridges. Among the  
799 various findings of the research, the radar measurements provided valuable structural information on the bridge decks at  
800 the joint connection between consecutive spans. Stryk et al. (2013) used a two-channel GPR system to provide accurate  
801 measurements of the dowel and tie bar positions in a concrete pavement. Analyses of the GPR outputs in terms of the  
802 translations of a rebar layer observed along the longitudinal axis allowed to identify an incorrect cut at the contraction  
803 joints. Alani and Tosti (2018) reported a structural detailing of an “immersed tube” major tunnel type connecting a number  
804 of segments at immersion joint points. Two sets of antenna systems with frequencies of 900 MHz and 2 GHz were used  
805 to establish structural details of the tunnel roof at the immersion joints. The data provided ample information confirming  
806 rather doubted construction design drawings/plans originally produced. The results obtained were conclusive in terms of  
807 construction materials and structural design configurations (shape and dimensions) as well as the identification of rebar  
808 positions at all the inspected immersion joint locations.

809

## 810 5. NEW METHODOLOGICAL AND DATA PROCESSING PROSPECTS FOR THE ASSESSMENT OF 811 REINFORCED CONCRETE STRUCTURES USING GPR: A CASE STUDY

812

813 A higher accuracy in rebar location and an evaluation of the concrete cover can be achieved using dedicated data  
814 processing algorithms, such as migration (Chang et al. 2009), and data interpretation methods such as hyperbola fitting  
815 (Tillard and Dubois, 1995; Shihab and Al-Nuaimy, 2005). Within this framework, although a suitable level of accuracy  
816 can be reached with the available commercial software, higher precision could be required for quality control and routine  
817 inspection purposes. To speed up the data processing stage and achieve faster results, a few velocity values of propagation  
818 are usually extracted randomly from a sequence of rebar reflectors in a B-scan. This is achieved by fitting the reflections  
819 from the clearest hyperbola shapes. An average velocity is then assigned to the whole area in order to obtain a map of  
820 migrated points. In this regard, relating an average velocity value to an entire investigation area is a suitable approach for  
821 the location of rebars in relatively small reinforced concrete areas, although it might not work for large and complex  
822 environments (i.e. investigation sites with a high variation of wave propagation velocity, which is mostly unknown). The  
823 process can be improved by sampling a consistent number of targets for an estimation of velocity by the hyperbola fitting  
824 method. Accordingly, a more representative propagation velocity value can be assigned to the area for data migration  
825 purposes. In view of this, a sequence of migrated B-scans collected using a typical acquisition grid can provide high-  
826 quality C-scans and contribute to a more accurate rebar location and evaluation of the concrete cover.

827 Within this context, the main aim of this research is to comprehend the effect of different data sampling percentages on  
828 the estimation of a wave propagation velocity value in a reinforced-concrete-paved area with a high-dense grid mesh of  
829 rebars.

830 An estimation of this parameter is therefore used for data migration purposes in order to produce a viable C-scan map for  
831 a more accurate location of concrete rebars.

832 To achieve this aim, the following objectives are identified:

- 833 ● to create a 2-D matrix of wave propagation velocity values estimated by hyperbola fitting of reflections from rebars  
834 over a whole inspected area;
- 835 ● to analyse the statistical distribution of the estimated propagation velocity values and calculate the data dispersion  
836 throughout the area;
- 837 ● to develop a data sampling methodology for the identification of a value of velocity of propagation representative of  
838 the entire area. This is sought by means of comparison between tomographic maps migrated at different sampling  
839 percentages (combination of longitudinal (L)/transversal (T) acquisitions and horizontal (HH)/vertical (VV)  
840 polarisations).



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## 842 5.1. Methodology

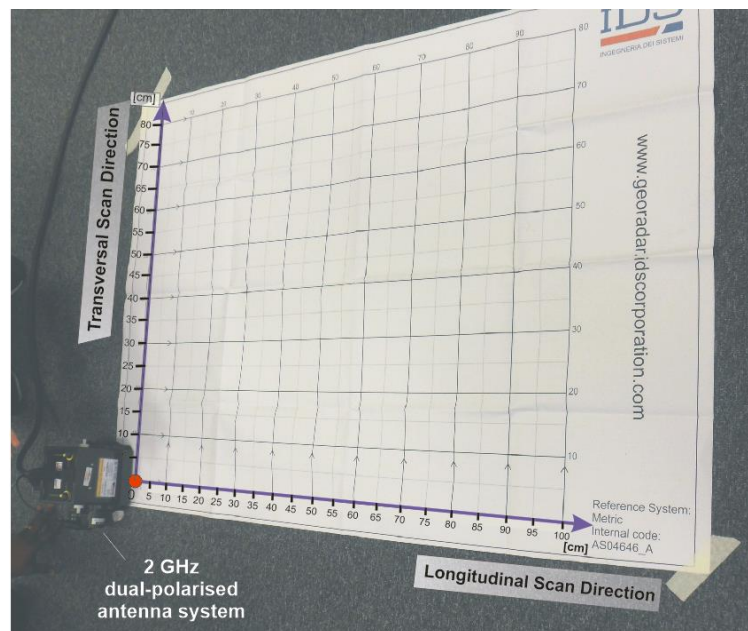
843 A multi-stage methodology is proposed based on four main chronological steps. The data acquisition is first carried out.  
844 A second main stage is focused on the estimation of the wave propagation velocity values using the hyperbola fitting  
845 method. This allows to create a 2-D matrix of data, with dimensions of the number of rebars detected throughout the  
846 inspection area. A third stage is developed to analyse the statistical distribution of the above estimates as well as the data  
847 dispersion. Finally, a data sampling methodology is proposed to extract a representative wave propagation velocity value  
848 for data migration purposes.

849

### 850 5.1.1. Data Acquisition

851 Data were collected on a reinforced-concrete-paved area (rebar density of  $\sim 5$  rebars/m<sup>2</sup>) with dimensions of 1m×0.80m  
852 (Fig. 15). The IDS Aladdin GPR system equipped with a dual-polarised antenna of 2 GHz central frequency was used for  
853 testing purposes. The antenna configuration allowed to collect two different sets of data on a single scan line, as per the  
854 HH and VV polarisations. GPR signals were acquired with a horizontal resolution of 1 cm, using a time window of 32 ns  
855 and 512 samples. Longitudinal and transversal scans were performed with a scan spacing of 5 cm.

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857

858 **Fig. 15** Scanning grid and 2-GHz dual-polarised antenna system used for data collection purposes

859

### 860 5.1.2. Wave Velocity of Propagation Data Matrix by the Hyperbola Fitting Method

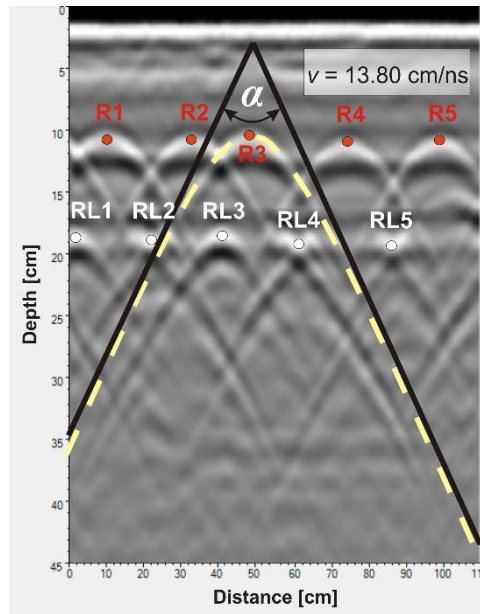
861 In view of the high-dense grid mesh of rebars and the relatively small scan spacing, the hyperbola fitting method (Shihab  
 862 and Al-Nuaimy, 2005) was used to estimate the wave propagation velocity data over the whole inspected area. In this  
 863 method, the wave velocity  $v$  is proportional to an angle  $\alpha$ , formed between the hyperbola asymptotes, according to the  
 864 following relationship:

$$865 \quad v = 2tg\left(\frac{\alpha}{2}\right) \quad (8)$$

866 Several studies in the literature have proven the dependency of  $v$  on the vertex coordinates, the target radius and the time  
 867 delay of the signal reflection. An example of application of the hyperbola fitting method to one of the rebars investigated  
 868 in this study is reported in Fig. 16.

869 From the analysis of the GPR data collected on site, two main layers of rebars were observed. A number of  $n$  upper lines  
 870 of rebar (being  $n_{max} = 5$ ) (i.e., R1, R2, R3, R4 and R5 in Fig. 16) overlying a number of  $l$  lower lines of rebar (being  $l_{max}$   
 871  $= 5$ ) (i.e., RL1, RL2, RL3, RL4 and RL5 in Fig. 16) were collected throughout the inspection area. For the purposes of  
 872 this study, only the upper rebars were taken into account. Hence, considering a scan spacing of 5 cm,  $m$  longitudinal  
 873 scanning lines of 1m length (being  $m_{max} = 17$ ) were collected across the 80cm-wide transversal dimension of the inspection  
 874 area (Fig. 15). This turned out to provide a matrix of rebar points  $\mathbf{P}_{ij}$  sized  $n \times m = 85$  (where  $i$  stands for the “scan  
 875 direction” (L, T),  $j$  stands for the “antenna orientation” (HH, VV)) and, hence, a matrix  $\mathbf{V}_{ij}$  of wave propagation velocity  
 876 data with an analogous size.

877



878

879 **Fig. 16** Wave propagation velocity estimation on a longitudinal scan (HH polarisation). Application of the hyperbola  
 880 fitting method and the B-scan layout of rebars within the inspection area (data displayed with a default  $v = 10$  cm/ns)  
 881 (Tosti et al., 2018)

882

### 883 5.1.3. Statistical Distribution Analysis

884 An analysis of the statistical distribution of the dataset of propagation velocity values  $v_{i,j,k}^{[m,n]}$  (where  $k$  stands for “the  
885 percentage of data collected within the inspection area”) is carried out to evaluate the data dispersion within the  
886 investigated area. The average value of velocity of propagation of the entire population  $\overline{v_{i,j,100}^{[m,n]}}$  (value averaged over  $k =$   
887 100% of the  $m \times n$  data in the propagation velocity matrix  $\mathbf{V}_{i,j}$ ) is taken as the benchmark for every combination of the  $i^{\text{th}}$   
888 scan direction and the  $j^{\text{th}}$  antenna orientation for the calculation of the residuals. Hence, the percentage residual at a  
889 random position  $[m, n]$  in the matrix  $\mathbf{V}_{i,j}$  is calculated as follows:

$$890 \quad \xi_{i,j,100}^{[m,n]} = \frac{v_{i,j,100}^{[m,n]} - \overline{v_{i,j,100}^{[m,n]}}}{\overline{v_{i,j,100}^{[m,n]}}} \times 100 \quad (9)$$

891 A matrix  $\mathbf{R}_{i,j}$  of residuals  $\xi_{i,j,100}^{[m,n]}$  sized  $m \times n$  can be therefore computed working out the values of  $v_{i,j,100}^{[m,n]}$  in Equation (8).

892 To evaluate the dispersion of propagation velocity values  $v_{i,j,100}^{[m,n]}$  against the benchmark value  $\overline{v_{i,j,100}^{[m,n]}}$ , the  $m^{\text{th}}$  average  
893 residual  $\overline{\xi_{i,j,100}^{[m,n]}}$  is defined along the  $m = 17$  scanning lines as follows:

$$894 \quad \overline{\xi_{i,j,100}^{[m,n]}} = \sum_{n=1}^{n_{\max}} \xi_{i,j,100}^{[m,n]} / n_{\max} \quad \forall m \in \mathbf{R}_{i,j} \quad (10)$$

895 and compared to the average residual of the entire matrix population  $\overline{\xi_{i,j,100}^{[m,n]}}$  (value averaged over  $k = 100\%$  of the  $m \times n$   
896 residuals  $\xi_{i,j,100}^{[m,n]}$  in  $\mathbf{R}_{i,j}$ ). Hence, data dispersion from this benchmark value of residuals can be interpreted as a measure of  
897 non-homogeneous distribution of the propagation velocity throughout the inspection area. It is also worthy of mention  
898 that this statistical analysis can be used to identify potential misalignments within a line of rebars. The frequency density

899 distributions of residuals for i) the entire matrix population  $\xi_{i,j,100}^{[m \times n]}$  and ii) every  $n^{\text{th}}$  rebar  $\overline{\xi_{i,j,100}^{[m,n]}}$  is also defined, being this  
900 latter expressed as follows:

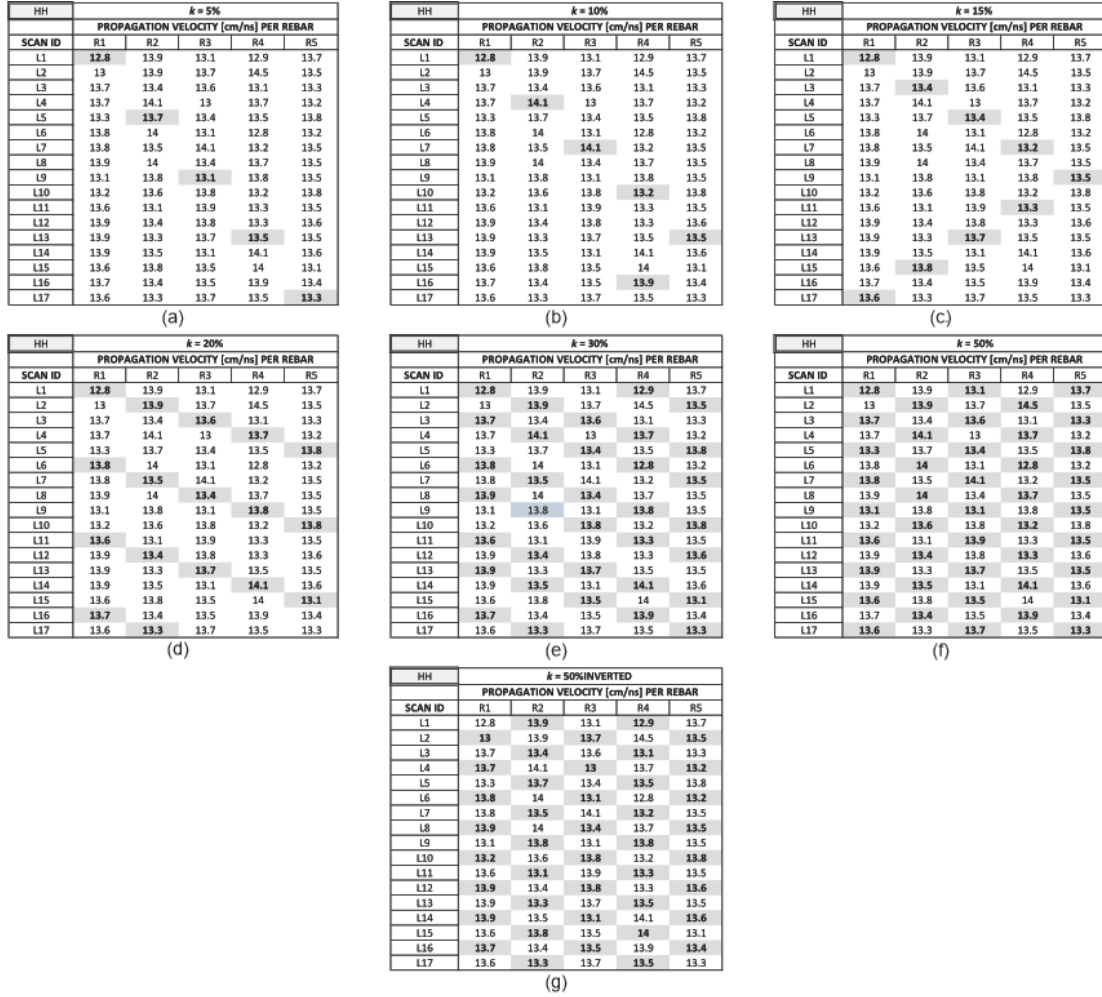
$$901 \quad \overline{\xi_{i,j,100}^{[m,n]}} = \sum_{m=1}^{m_{\max}} \xi_{i,j,100}^{[m,n]} / m_{\max} \quad \forall n \in \mathbf{R}_{i,j} \quad (11)$$

902

### 903 5.1.4. Data Sampling and Migration

904 A number of different  $k^{\text{th}}$  percentages of  $v_{i,j,k}^{[m,n]}$  values (with  $k = 5, 10, 15, 20, 30, 50, 50\text{inverted}$ ) is sampled over the  $n \times$   
905  $m$  data of  $\mathbf{V}_{i,j}$ . Sampling is developed to ensure an evenly distribution of data across the longitudinal and transversal

directions of the area (Fig. 17). The sampling configuration ranges from diagonal (i.e., smallest percentage case (Fig. 17a) to a chessboard-like arrangement at  $k = 50$  and 50inverted (Fig. 17f-g). The optimum sampling rate of wave propagation velocity is therefore assessed by way of comparison between the migrated maps.



**Fig. 17** Data sampling (grey cells) layout of the  $k^{\text{th}}$  percentages of  $v_{i,j,k}^{[n,m]}$  values in  $\mathbf{V}_{ij}$  (with  $i = L$ ;  $j = \text{HH}$ ;  $k = 5$  (a), 10 (b), 15 (c), 20 (d), 30 (e), 50 (f) and 50inverted (g))

### 5.1.5. Results and Short Discussion

#### Statistical Distribution Analysis

Table 4 shows the main statistics from the wave velocity of propagation data matrices  $\mathbf{V}_{ij}$ , in regard to the full set of available combinations between  $i$  (L, T) and  $j$  (HH, VV). Overall, the average value of propagation velocity  $\overline{v_{i,j,100}}$  is observed to vary between 13.43 and 14.22 cm/ns across the four  $i^{\text{th}}, j^{\text{th}}$  combinations. The maximum values for the standard

919 deviation ( $\sigma_{v_{i,j,100}}^{\max} = 1.03$  cm/ns) and the standard error ( $\sigma_{v_{i,j,100}}^{\max} = 0.11$  cm/ns) are both observed in the case of  $i = L$  and  $j$   
 920  $= VV$ .

921

922 **Table 4** Main statistics from the wave velocity of propagation data matrices  $V_{i,j}$

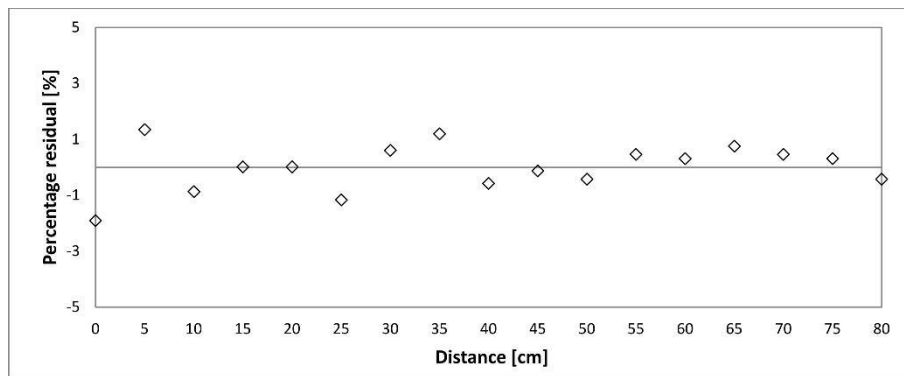
Statistic Parameter [cm/ns]	Longitudinal Scan		Transversal Scan	
	$(i = L)$		$(i = T)$	
	HH	VV	HH	VV
	Polarisation	Polarisation	Polarisation	Polarisation
	$(j = HH)$	$(j = VV)$	$(j = HH)$	$(j = VV)$
$\overline{v_{i,j,100}}$	13.54	13.58	13.43	14.22
$\sigma_{v_{i,j,100}}$	0.04	0.11	0.10	0.06
$\sigma_{v_{i,j,100}}$	0.33	1.03	0.78	0.46
$v_{i,j,100}^{\min}$	12.80	9.10	9.90	13.50
$v_{i,j,100}^{\max}$	14.50	14.80	14.10	15.40

923

924 In regard to the statistics for the residual distributions, data from the longitudinal scan L / HH polarisation ( $i = L; j = HH$ )

925 are reported in Table 5. Fig. 18 and Fig. 19 show the distribution of the average  $\overline{\xi_{i,j,100}^{[m,n]}}$  and the  $n^{\text{th}}$  residual  $\xi_{i,j,100}^{[m,n]}$ ,  
 926 respectively, from the scan position  $m = 1$  (0 cm on the Tscan axis) to  $m = 17$  (80 cm on the Tscan axis), sorted by the  $n^{\text{th}}$   
 927 upper line of rebars.

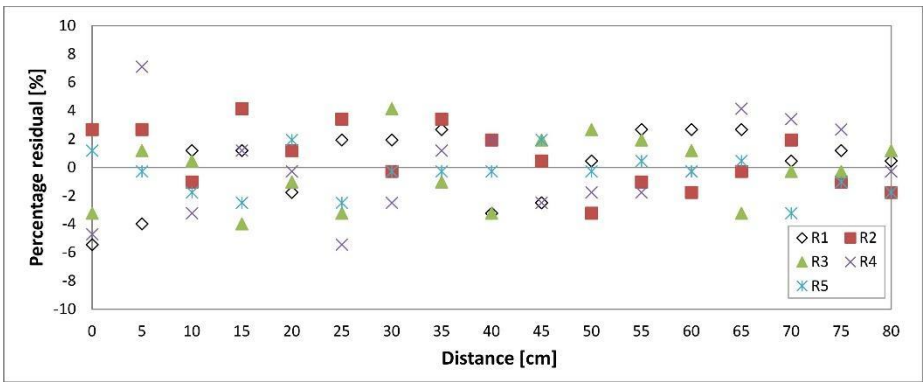
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930 **Fig. 18** Trend of the average percentage residual  $\overline{\xi_{i,j,100}^{[m,n]}}$  along the  $m = 17$  scan lines (Tosti et al., 2018)

931



932

933 **Fig. 19** Trend of the  $n^{\text{th}}$  percentage residual  $\xi_{i,j,100}^{[m,n]}$  along the  $m = 17$  scan lines (Tosti et al., 2018)

934

935 From the analysis of  $\overline{\xi_{i,j,100}^{[m,n]}}$  (Fig. 18), a larger dispersion is observed from  $m = 1$  (0 cm on the Tscan axis) to  $m = 7$  (35

936 cm on the Tscan axis), as opposed to the rest of the scans. From Fig. 19, it can be noticed that the first line ( $n = 1$ ; i.e., R1

937 in Fig. 16) and the fourth line ( $n = 4$ ; i.e., R4 in Fig. 16) of rebars show the largest dispersion among the five lines

938 investigated. On the contrary, the fifth line of rebar ( $n = 5$ ; i.e., R5 in Fig. 16) has a lower variation over the benchmark

939  $\overline{V_{i,j,100}}$ , with  $i = L$  and  $j = HH$ . This evidence is confirmed by the percentage residuals  $\xi_{i,j,100}^{[m,n]}$  calculated at each  $n^{\text{th}}$  rebar

940 in Table 5, where the rebar line R5 has the lowest statistics.

941

942 **Table 5** Main statistics on the percentage residuals  $\xi_{i,j,100}^{[m,n]}$  at each rebar

Statistic Parameter	Longitudinal Scan ( $i = L$ )				
	HH Polarisation ( $j = HH$ )				
	R1	R2	R3	R4	R5
	( $n = 1$ )	( $n = 2$ )	( $n = 3$ )	( $n = 4$ )	( $n = 5$ )
$\overline{\xi_{i,j,100}^{[m,n]}}$	0.16	0.68	-0.28	-0.06	-0.50
$\sigma_{\xi_{i,j,100}^{[m,n]}}$	0.62	0.53	0.59	0.80	0.36
$\sigma_{\xi_{i,j,100}^{[m,n]}}$	2.57	2.18	2.44	3.30	1.49
$\xi_{i,j,100}^{[m,n]\min}$	-5.45	-3.23	-3.97	-5.45	-3.23

$\xi_{i,j,100}^{[m,n]\max}$	2.68	4.15	4.15	7.11	1.94
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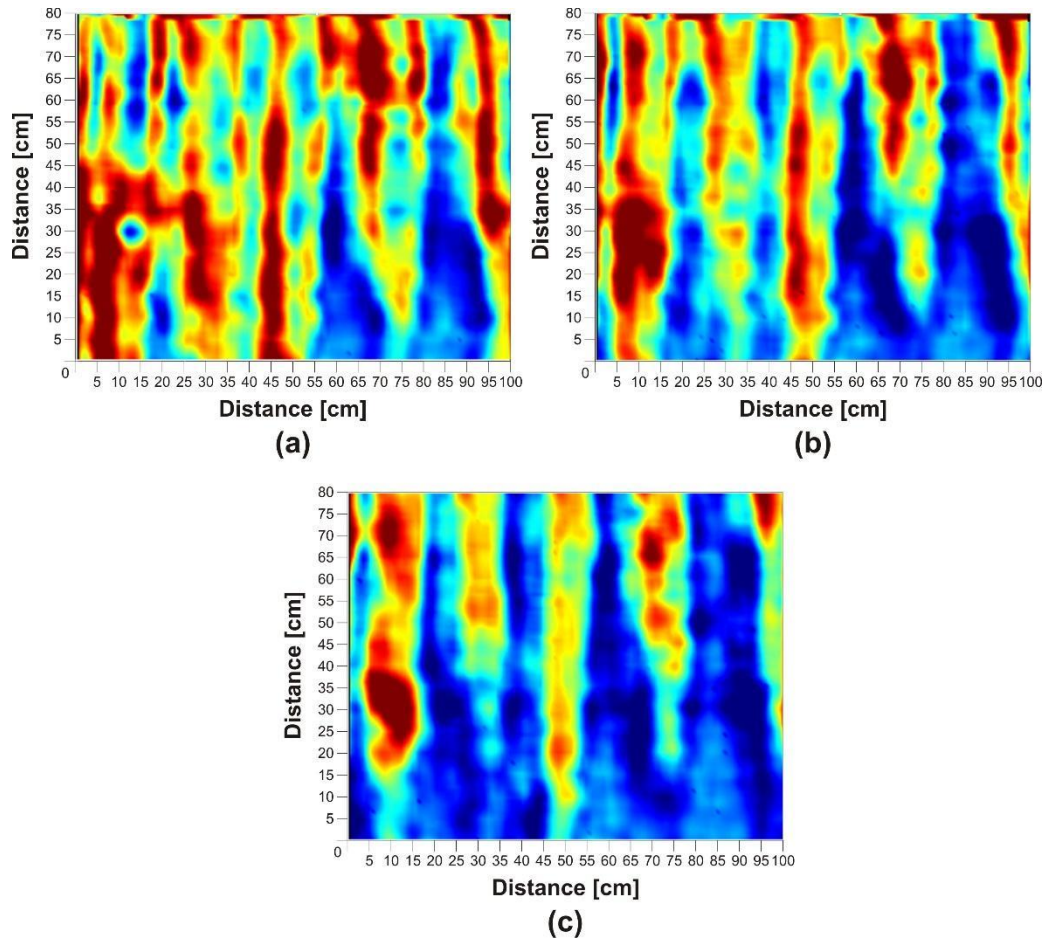
943

#### 944 *Data Sampling and Migration*

945 Data migration was applied using the average values of velocity of propagation  $\overline{v_{i,j,k}^{[m,n]}}$  at  $k = 5, 10, 15, 20, 30, 50$  and  
946 50inverted. To analyse the viability of the proposed sampling approach and to identify the optimal percentage of data to  
947 be used for migration purposes, the corresponding migrated C-scan tomographic maps were compared. Fig. 20 shows the  
948 C-scan tomographic maps of the inspection area obtained using data from the longitudinal scans L and the HH polarisation  
949 ( $i = L; j = HH$ ). The maps are represented at a depth  $z = 13$  cm after applying data migration at  $k = 10, k = 30$ , and  $k =$   
950 100.

951 It can be noticed that the use of a small percentage of sample data (i.e., Fig. 20a), is not sufficient to reproduce effectively  
952 the rebar configuration, as opposed to the results shown in Fig. 20c. On the other hand, the spatial sampling and the data  
953 percentage used in the case of  $k = 30$  (i.e., Fig. 20b), indicate that a more consistent output is provided. This can turn out  
954 to provide a more accurate location of the reinforcement bars as well as a better estimation of the concrete cover in  
955 reinforced concrete structures.

956



**Fig. 20** C-scan maps of the area inspected (depth  $z = 13$  cm) after the application of data migration at different  $k^{\text{th}}$  percentages of sampling. **a**  $k = 10$ , **b**  $k = 30$ , **c**  $k = 100$  (Tosti et al., 2018)

## 6. CONCLUSIONS AND FUTURE PROSPECTS

In this paper, an overview of the existing literature within the subject area of Ground-Penetrating Radar (GPR) methods for the investigation of reinforced concrete structures is reported. Six major application areas have been identified where experimental, numerical and theoretical research on GPR has been developed. The review demonstrates that the applications of GPR to reinforced concrete structures are continuously growing. It was also observed that research in some application areas has been mostly or exclusively carried out at the laboratory scale and, similarly, some application areas have been investigated on real-life structures only. In terms of types of GPR systems used for purpose, existing research has been predominantly developed using pulsed-radar systems, as opposed to stepped-frequency continuous-wave (SFCW) radar systems. This was seen as being due to the higher cost of the electronics, an overall higher complexity in data processing (i.e. high computational requirements) and a lack of dedicated commercial software. Overall, ground-coupled antenna systems working in a range of nominal frequencies between 1 GHz and 2 GHz were observed to have been used across all the application areas. High-frequency GPR systems were in fact demonstrated to present a good



973 trade-off between the resolution and the depth of investigation necessary to meet the requirements of the identified  
974 application areas.

975 Automation and the need for faster processing methods seem to be a major focus of the research, especially over the last  
976 decade. The common high density and rather regular arrangement of rebars along with the advances achieved in  
977 computational methods, have fostered the development of neural network and machine learning-based approaches. Within  
978 this framework, algorithms based on full-waveform inversion models seem to be viable research avenues to pursue in the  
979 near future.

980 Most common and long-standing applications have been focused on the location of rebars along with the evaluation of  
981 the concrete cover, all of which were predominantly developed on real-life structures. This is due to the relatively  
982 straightforward operations and data processing required to achieve the application goals. Research on the rebar corrosion  
983 and on the assessment of the concrete characteristics by dielectric properties has been mostly and totally carried out in a  
984 laboratory environment, respectively. This could be related to practical constraints not allowing for full control of the  
985 boundary conditions, as well as to the difficulty of validating research outcomes by a destructive way of practice. On the  
986 contrary, these factors are easier to control and analyse in a laboratory environment. In addition, it is worth mentioning  
987 that most of the research on the dielectric properties of concrete was carried out in the 2000s for approximatively a decade.

988 The analysis of the literature review has emphasised that the estimation of the rebar size is a topical and very challenging  
989 application area. Research has been performed exclusively at the laboratory scale, proving that applicability in this area  
990 is at an early stage of development. A common approach is to analyse the problem using numerical modelling and validate  
991 the observations with experiments carried out on samples in a laboratory environment. It was also highlighted that  
992 structural detailing for the assessment of the integrity of reinforced concrete structures has gained momentum over the  
993 past decade. To this effect, GPR has been demonstrated to be a viable method to achieve an early diagnosis of decay in  
994 important areas, such as anchorages in building foundations and pre-stressed beams, joints in bridge decks, and tunnel  
995 linings.

996 A case study on a new methodological and data processing approach for the assessment of reinforced concrete structures  
997 with GPR is discussed in the final part of the paper. A data sampling investigation for the estimation of a propagation  
998 velocity value representative of a reinforced-concrete-paved area with a high-density grid mesh of rebars is proposed. To  
999 this purpose, a high-frequency GPR system with a 2 GHz dual-polarised antenna (HH/VV) was used and longitudinal and  
1000 transversal scans were collected. A multi-stage methodology including i) data acquisition, ii) use of the hyperbola fitting  
1001 method, iii) a statistical distribution analysis of the velocity values and residuals, and iv) a data sampling methodology  
1002 and migration, was proposed. Results have demonstrated the advantages of using a dual-polarised GPR system in order  
1003 to improve the detectability of rebars, including secondary bottom lines of reinforcement. The horizontal polarisation was

1004 proven to be more stable compared to the vertical one. Finally, it was demonstrated that a more accurate visualisation of  
1005 the rebars can be obtained using 5 cm scan spacing and information from at least 30% of the targets in a high-density grid  
1006 mesh arrangement of rebars.

1007

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